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CONTENTS

April, 1960

Papers

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

CONSERVANCY DISTRICTS AS FLOOD CONTROL ORGANIZATIONS[a]

By Cloyde C. Chambers,[1] F. ASCE

SYNOPSIS

A well designed conservancy district is a valuable organization for de-loping, constructing and administering a comprehensive water management ogram for an intrastate watershed.

The evidence, as provided by many operating districts, indicates that this pe organization is well adapted to the solution of flood problems, especially ien approached on a drainage area basis. It is most applicable in the field at lies between those problems that can be solved best be existing local vernmental agencies and the problems of the major rivers, which are inter-ate in character and in which a federal interest is predominant.

Some of the most successful flood control projects have been achieved by nservancy districts which have kept a large degree of control of the plan-ng and operation at the local level and which have assumed a fair share of e project costs. Local project control and maintenance can be provided best there is a conservancy district or similar organization with the legal thority and financial ability to participate in the project, even though there e large state and federal interests involved.

A conservancy district, to operate successfully, must be a political sub-vision of the state, organized on a problem area basis. It must be organized the request of and have the support of the local people. It must have thority to plan and to execute plans. It must have financial responsibility d the power to levy taxes and assessments. It must have authority to ac-ire property, including the right of eminent domain, and power to establish

Note.—Discussion open until September 1, 1960. To extend the closing date one onth, a written request must be filed with the Executive Secretary, ASCE. This paper part of the copyrighted Journal of the Hydraulics Division, Proceedings of the Amer-an Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

a Presented at the May, 1959 ASCE Convention in Cleveland, Ohio.

1 Cons. Engr., Dayton, Ohio.

and enforce rules and regulations. It must, also, have authority to cooperat with other levels of government and their agencies.

INTRODUCTION

The engineer engaged in public works must accomplish his goals throug the medium of diverse types of public agencies and organizations which ma have a variety of legal powers, administrative authorizations and objective: The organizational means and available legal powers for solving his problem and constructing his works are, therefore, as important to him as the ma terials, tools and equipment he selects or designs. He should have a knowl edge of the advantages and disadvantages of the various types of public organi zations available for his use. This is particularly true in dealing with floc control problems, each one of which is generally unique in character. Hi recommendations, as to the provisions to be contained in legislation creatir the powers, authority and organization with which he will have to work, shou be given much weight, because of the effect the legislation will have on th quality of the work to be produced.

In drafting legislation for flood control purposes, it is important that prc visions for the administrative organization be designed to be adaptable to variety of flood control problems. The enactment of the first enabling legis lation, making possible the creation of conservancy districts, required a much pioneering as the technical problems involved in the design of projec to be carried out under the legislation. Negligence in either case would ha increased the chance of failure.

USE OF TERMS

The term "conservancy district," for the purposes of this paper, is a plied to a variety of special local organizations created by legislation a special taxing and assessment districts with designated powers and some lo cal financial responsibility.

The term "flood control" is interpreted to cover the broad field of meas ures that can be used to prevent or mitigate losses and other ill effects, whi normally occur as a result of heavy precipitation. These measures can b grouped, roughly, into three main classifications (the first has a tendency merge with the second):

1. Flood prevention measures, which include, among others, those meas ures for preventing or retarding the runoff of excess water from the land a for retarding the flow of small streams or brooks.

2. Flood control measures, consisting principally of works that are d signed to control the flow of flood waters by the temporary storage of exce water in retarding basins or reservoirs, the structures of which can have wide range in size.

3. Flood protection measures include local works at critical or important oints in the flood plains, such as channel improvements, protective levees, alls, etc., either as a sole remedy or for supplementing one or both of the ther types of measures.

CONSERVANCY DISTRICTS

Conservancy districts vary in organization, powers and objectives, in the everal states where such districts are found, because the wording of the espective enabling legislations, and the authorities given to the districts are uite different. In some cases, this variation is the result of differences in ie state constitutions and the existing laws to which the districts must con- orm. In other instances, the conditions and problems to be met are not the ame. States with semi-arid areas may give irrigation a prominent place long with flood control, as, for example, the New Mexico law under which the liddle Rio Grande Conservancy District has operated. Power may be given nportance in others, as in the New York law, under which the Hudson River egulating District operates.

Some of the provisions, such as method of organizing, assessing for bene- ts, etc., that are now incorporated in conservancy acts have been used for iany years in other enabling legislation for the solution of flood problems. aws for the creation of levee and drainage districts date back to the days of ie early settlements of the country. These earlier laws provided for dis- icts with a limited scope of activities and frequently authorized project con- ruction without adequate provisions for maintenance. The boundaries of ich districts were often limited by lines dividing other existing political ibdivisions. The enabling legislation was frequently drawn with a particular oblem in mind, consequently it did not fit other problems, or it lacked some ements that would have provided for good flexible operation. Eventually, the eed for legislation with a broader viewpoint, and with provisions for coordi- ting a variety of related problems, became apparent.

Flood problems and their solutions are generally unique. It is not often at blueprints, successfully used at one location, can be moved bodily to iother problem area and achieve the same degree of success.

A conservancy district law should be broad enough to permit a complete id honest inventory of the flood problems in the various parts of the district id to give freedom to ferret out and explore the various possibilities for the lution of the problems. The best solution for major flood problems is ually found in a variety of measures rather than in a single one.

The need for an organization that can fill the gap between those problems at can be solved by existing local agencies and the interstate problems on r larger river systems, in which the federal government has an interest, s been emphasized by the interest shown in conservancy districts in Ohio d other states. In addition, it has been shown that conservancy districts are metimes useful in the solution of the local flood problems and the problems river systems, such as the Muskingum watershed, which are tributary to e larger systems, such as the Ohio and Mississippi Rivers.

It is rare that existing political subdivisions have the facilities, authority d flexibility of action necessary to cope even satisfactorily with local prob- ms. It is not often that the boundaries of flood problem areas are common the boundaries of such subdivisions. Flood control and water use problems

tend to divide themselves into stream drainage areas. The Miami Co servancy District and the Muskingum Watershed Conservancy District a examples of situations in which a group of several communities in one drai age area have been given an opportunity to pool their problems for unifi solution. Satisfactory solutions for the problems in those districts would ha been impossible, had such an opportunity not been created.

There are examples of communities electing to stay out of conservan districts covering large drainage areas in order to organize their own sm districts to provide for their own local protection. Such protection has ge erally been less adequate than that which would have been provided by t larger district. As a result, these communities, as, for example, Mt. Vern Ohio, in the Muskingum area, and Springfield, Ohio, in the Miami area, ha suffered repeated, serious flood losses because of inadequate protection.

OHIO EXPERIENCE

Interest in enabling legislation for the creation of conservancy distri first became active in Ohio, immediately following the floods of March, 19 The enactment of the Ohio conservancy act was prompted by a realization th in some cases, it would be desirable to work out flood problems on a riv drainage-wide basis, rather than solely by individual local projects, and permit a group of communities in one valley to pool their problems for unif solution. Without such enabling legislation it seemed impossible to solve so of the flood problems of the state. The persistent and well-guided determin tion of the people in the Miami Valley to do something about the flood probl in that area gave strong impetus to the enactment of the first Ohio conse vancy act. It was a case of a new invention being developed to meet a new a urgent need.

The acceptance of the conservancy type organization for flood control a allied problems has been more pronounced in Ohio than in some other sta because of the early recognition of the need for such an organization. greater use in that state has been influenced to some extent by the success the Miami Conservancy District. The Miami project was financed with fu derived from assessments on appraised benefits to property in the distri The original cost of the project has now been paid in full. The Upper Sci Drainage and Conservancy District, the first to be organized in Ohio, has a been financed solely with assessments levied on benefited property. Both d tricts have remained active and have provided a high standard of maintena for their projects with funds raised through maintenance assessments property receiving project benefits.

A total of twenty-two districts and four sub-districts have been organi under the Ohio act. They range in size from a district involving a small gr of farmers to one that includes about one-fifth of the state. Fourteen of twenty-two districts have included flood control along stream valleys as of their principal purposes. Two others include flood control and beach e sion prevention along the shore of Lake Erie, as their prime objectives.

Of the fourteen districts that included flood control along stream valleys a principal purpose, seven have been completed or are in the process of c structing their initial programs; two are in the development stage; one is active; and four are either in the process of being dissolved or have been d solved.

Some districts have become inactive or disorganized because it was found hat the estimated costs of the proposed work either exceeded the estimated enefits to be derived therefrom, or were higher than the people would accept. n some instances, it was found that the proposed plans did not offer a suitable or acceptable solution to the problems, and in others, the districts were romoted without conducting a thorough and honest educational program among he local people most vitally affected by the proposed program.

COMPONENTS OF GOOD CONSERVANCY LAWS

Current thinking on the development, management and control of the naion's water resources requires that the enabling legislation for the creation f conservancy districts provide for a broad view of the complicated and inerrelated water, soil and sanitary problems, and for considerable latitude in uthority for solving such problems.

As conservancy districts generally do not coincide with the usual political oundaries, it is necessary to provide for their establishment through special egislation for creating a separate political subdivision of a state. Such a type f organization is also necessary because of taxing and other powers that disricts must have in order to operate.

The enabling acts must provide for the free expression of the desires of the eople and for an opportunity for them to accept or reject any proposed disrict. Objections to conservancy districts are very common and arise for nany reasons.

Some conservancy district enabling laws have been criticized on the rounds that they are too long or complicated (the Ohio law, for example) and hat the steps required for development and operation of a program are too umerous and time consuming, especially for small or local projects. On the ther hand, attempts at brevity generally omit some one or more essential lements of a workable law. In addition, after a proposed development passes hrough all the checks and balances of a well-drafted conservancy district law, ts soundness is fairly well established from a practical as well as a technial viewpoint.

Sometimes, objections to conservancy acts are made because the acts are udged by the provisions of individual sections rather than as an act composed f many interrelated working parts. Objections are also raised over the use f a conservancy district for the solution of local flood problems because the ork can be done through existing governmental agencies, thus avoiding the reation of a new organization.

There are some advantages in the use of conservancy districts for some ocal problems. As previously mentioned, boundaries of existing agencies do ot often fit desirable boundaries for flood problem solutions. In addition, the nanaging directors of a district usually serve out of a civic interest or duty ather than for any financial or political gain. Their attention is directed toard a single objective and its allied features, and they are not burdened with ther day-to-day routine activities.

The degree of influence a conservancy district may have on the solution of ood control and allied problems may depend, in some cases, upon the poliies of the state and federal governments at the time the project is undertaken.

At the time when the conservancy district idea began to be developed, the ctive interest of the federal government in flood control was confined largely

to the main rivers that had been or were being developed for general naviga tion. Then the field was open for conservancy districts to operate under th original concept of a self-financing organization.

Changes in federal and state policies, with respect to flood control sinc the conservancy district idea was first brought into prominent use, especiall during the past quarter of a century, have tended to change the function of suc districts, but they have not entirely eliminated the need for them. Many of th newer districts are being established to cooperate on federal projects rathe than to operate independently.

Persons who believe that the entire development, planning, design, con struction, maintenance and operation of all flood control projects, regardles of type and size, are the sole responsibility of federal and state government may find little use for conservancy districts as flood control organization The writer, who is an advocate of retaining a high degree of local respons bility and control, does not subscribe to that belief.

In some river drainage areas, it is unwise to divorce the flood problem in the area from the needs for the development and wise use of the water re sources of that area. In other areas, the water problems are tied closely the problems of good management of other natural resources. A conservanc district created for purposes that will include all related and associated prob lems is in a better position to develop or guide the development of a compre hensive plan that will have a broad scope and balance between functions tha is an organization that has a predominant interest in only one phase of th over-all problem.

Many valleys, where conservancy districts are organized for flood contr purposes, are experiencing an increasingly rapid and complex econom growth. These developments create water, sanitation, highway and othe problems, many of which have important relationships with the work of th districts. In some states, very moderate amendments to their conservanc acts would enable conservancy districts to better serve those communiti having such closely related problems. At least, the conservancy distri might be the organization for dealing with all water problems, rather than wi a single purpose such as flood control.

There have been alterations, amendments, and, in some cases, redrafti of conservancy acts, where experience has dictated it to be necessary, in or der to meet changing or overlooked conditions. The original concept for con servancy districts was for flood control, with only rather general provisio for other conservation and water problems. The tendency in amendments ha been to expand and detail those provisions. Experience with conservancy di tricts has not suggested the need for radical changes in the original concep However, in some cases, there is a need for new powers or a new approach implement that original concept, as for example, how to assess or charge f water use in a ground-water replenishment project.

Conservancy districts should have some sort of zoning and regulating au thority, in order that serious flood problems can be prevented before th arise, and in order that encroachments and objectionable obstructions to flo flow are not built in the flood plains. This authority may be exercised, eith alone or in cooperation with some other proper agency, to restrict develo ments in flood plains until it is economically and otherwise feasible to provi adequate flood protection.

Most conservancy districts have the authority to move and reestabli communities that will be disturbed by the construction of district project

This authority should be broadened to permit moving communities out of flood plains, where it is economically and otherwise practical to do so, and where it is more feasible than protecting them against flooding.

In some cases, through the authority of conservancy districts, the pattern of railroad tracks has been changed to permit the construction of the most feasible plan for flood protection and, with a little forethought, at the same time, to provide a more efficient plan viewed from railroad and general welfare viewpoints. Examples are the work done by conservancy districts at Pueblo, Colorado and Massillon, Ohio.

In order to carry out its work, a conservancy district must be able to raise funds. This is usually accomplished either through an ad valorum tax on the taxable property in the district, by assessments made on the property in proportion to the benefits received, or both. These methods are used to pay for the preliminary expense of organizing the district, the preparation of the plan of works, the construction of the works and their operation and maintenance.

Some states have had difficulty in incorporating within their constitutional framework legislation that will provide an equitable and legal means of distributing the preliminary expense of organization of the district, preparing plans, making appraisals and other expenses prior to the receipt of funds from assessments on appraised benefits. Some suggest that these expenses be met with appropriations by the state. This is equitable only in case there is definite state interest in developing the possibilities of the proposed project and should be done in a manner that does not diminish local control over the affairs of the district. Preliminary funds should be made available promptly after the district is organized in order that planning can go forward while initial enthusiasm is high.

The problem of preliminary funds has been solved successfully in several cases by advances of funds or loans made by other agencies, such as counties, organizations, such as Chambers of Commerce, or individuals, with the understanding that such funds will be repaid when and if funds are made available from assessments on benefits or from other sources. The State of Indiana, for example, has made loans to conservancy districts for preliminary expenses from its Flood Control Revolving Fund. Financing in this manner is a test of the local appraisal of the merits of the proposed program and the sincerity of the local sponsors.

A conservancy district has an advantage over some other types of districts and some local governmental agencies in the provisions for maintenance of the works of the district. The right of the district to make annual maintenance assessments on benefited properties provides funds to keep the improvements well maintained. This helps to keep the works in a high state of efficiency at all times, and at less expense through the correction of maintenance needs while they are small instead of by large maintenance or rebuilding jobs at rare intervals. The contrast between the types of maintenance usually provided by a conservancy district and that of some existing local governmental agencies is often very striking.

The function of conservancy districts as organizations to raise funds for the construction, maintenance and operation of projects may have been reduced as federal and state appropriations have become increasingly available for such purposes. However, the need for such organizations to provide the local cooperation for many projects still exists.

An equitable subdivision of costs among the federal government, state and local interests, in proportion to their respective benefits, may be appropriate

in some cases. The appropriations by the federal government have tended to exceed their fair share of the cost. The tendency for states to make appropriations to reduce or eliminate local flood assessments is becoming more prevalent. The need for an alert local agency is as great now as it ever has been, not only as an agency to provide those funds required locally, but also to protect the local interests. There may seem to be little need for a conservancy district in the case of a system of federal flood control reservoirs to be constructed on a main tributary river system in the area but designed almost exclusively for benefits to be provided outside the area. However, a local organization, with the broad powers that a conservancy district should have, should be able to ferret out any alternate or amended plan which would give comparable outside benefits and, at the same time, be more desirable from a local viewpoint.

Federal participation in the cost of channel improvement, levee and other local projects is generally limited, under present federal policy, to the first construction cost. Some local agency must assume the responsibility for acquiring the necessary lands, rights-of-way and easements for constructing the project and must agree to maintain and operate it after it is built. Similar requirements must be met for projects in which the United States Department of Agriculture participates, under Federal Statute Public Law 566. These are proper functions of a conservancy district, and if administered by such an organization will insure the greatest value to the respective areas from the federal contribution.

A conservancy district is in a good position, also, to coordinate the activities of various agencies and units of government interested in flood control and other water problems in its area. It can provide the assurance and cooperation required on federal projects through one organization, and eliminate the need for the federal governments to enter into many agreements.

The broad powers generally vested in a conservancy district give opportunities to provide other benefits when local flood protection works are built through an urban area. With proper planning, unpleasant river fronts can be changed into the most attractive parts of a community. In some cases, the authority of conservancy districts should be broadened to permit more active cooperation in other community activities closely allied to flood control.

SUMMARY

A conservancy district, to serve present day needs, must be adaptable to a broad range of purposes. It is not enough that they provide for flood control alone; they must be able to cope with many other water problems and related activities.

Enabling legislation for the creation of a conservancy district should provide for at least the following features and powers:

1. It should be a political subdivision of the state, be created under state law, and should have perpetual existence except as provisions for its dissolution are provided in the enabling legislation.

2. The initiative for the creation of a conservancy district should come from local interested persons, corporations or political subdivisions by petition, vote or otherwise, and those opposed to it should have an opportunity to register and argue their opposition.

3. The purposes for which a district may be organized should include the prevention or control of floods and the development and wise use of the water resources of the district, in accordance with needs when such development can be accomplished to good advantage along with flood control.

4. It should have the authority to (1) develop, or cause to be developed, a plan for the improvements necessary to meet the purposes for which the district is created, (2) cause such a plan to be officially adopted after passing through the democratic processes necessary to give proper checks and balances, and (3) amend the plan from time to time, as needed.

5. It should have the authority, alone or in cooperation with other agencies, to construct, maintain and operate its works and improvements.

6. It should have the authority to establish and enforce the rules and regulations necessary to accomplish its purposes and protect its works.

7. It should have the authority to control, hold, and acquire by purchase, condemnation, donation or otherwise, all lands and rights necessary to construct, use and maintain its works.

8. It should have the right of eminent domain, including a dominant right over utilities and subordinate public corporations.

9. It should have the authority to secure by special assessments the funds necessary to carry out its purposes, and the authority to issue bonds in anticipation of their collection.

10. It should have the authority to enter into contracts or other arrangements with the United States and state governments, or any agencies thereof, and with persons and corporations, for cooperation and assistance in carrying out any of its purposes, objectives and authorities.

Like many other undertakings, the degree of success that can be expected from a conservancy district will depend largely upon how well it is administered. A conservancy district created under an ideal law can be only moderately satisfactory if it gets into incompetent or insincere hands. On the other hand, one created under a law that may lack some desirable features can be successful when used by a community with vision and administered by competent and sincere leadership. Such leadership exists in nearly every community. In any case, success will require sacrifices and services from people who will not receive their full reward in financial compensation.

The final worth of a water control and use project will depend upon how well society is able to adapt itself to the benefits the project offers. There have been projects of sound economics and engineering which have had disappointing results because the people in the area have been unable to use the opportunities offered to good advantage. Less attractive projects have become worthy because the people have adapted every possibility to good use.

The real satisfaction of the project builders does not come at the completion of the project, but rather when they go back after many years and find that the people have been able to take full advantage of the opportunities that were offered, and when they see that the test of time has proved their efforts to be worthwhile. The more the most directly benefited individuals have contributed toward the initiation, development, construction, maintenance and operation of a conservancy district project, the better chance there is for the realization of such satisfaction. A conservancy district can be a worthwhile tool for a community with foresight.

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A COMPARISON OF STREAM VELOCITY METERS

By F. Wayne Townsend[1] and F. A. Blust[2]

SYNOPSIS

A description of the U. S. Lake Survey method of making stream flow measurements is given, and comparisons of current velocities simultaneously measured by cup and screw type current meters in the lower Niagara River are presented. The conclusion is reached that the two types of meters give identical results in Lake Survey flow measurements.

INTRODUCTION

Among the responsibilities of the United States Lake Survey is the measurement of the flows in the outflow rivers of the Great Lakes, which include the St. Marys, St. Clair, Detroit, Niagara, and St. Lawrence rivers. Such measurements were first made by the Lake Survey in the 1860's and have been made many times since. Fig. 1 shows the Great Lakes and their outflow rivers. In recent years, vastly increasing hydro-electric power development and the agreements incident thereto between the United States and Canada for the sharing of the Great Lakes outflows have rendered the accurate determination of these flows more important than ever before.

Note.—Discussion open until September 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

1 U.S. Lake Survey, Corps of Engrs., U.S. Army, Detroit, Mich.
2 U.S. Lake Survey, Corps of Engrs., U.S. Army, Detroit, Mich.

Recent flow measuring practice by the Lake Survey has involved the use of cup-type current meters which respond to the water movement by means of a number of cups revolving around a common vertical axis. There is considerable evidence[3-9] that a revolving cup meter will over-register, that is, give a velocity higher than actual in turbulent water or as a result of movement of the meter. There is also evidence[3,4,5,6,8,9] that a screw-type current meter which responds to the water movement by means of a number of vanes revolving around a common horizontal axis does not have this tendency to over-register. Because of the present need for greatest possible accuracy of flow measurements, a comparison was made of current velocities obtained simultaneously by cup meters in present use and screw meters of latest design. The data on which this comparison was based were obtained under conditions typical of those encountered by the Lake Survey in measuring flows of the outflow rivers of the Great Lakes, and the comparison is intended to show only whether or not the two types of meters will give significantly different results under these conditions.

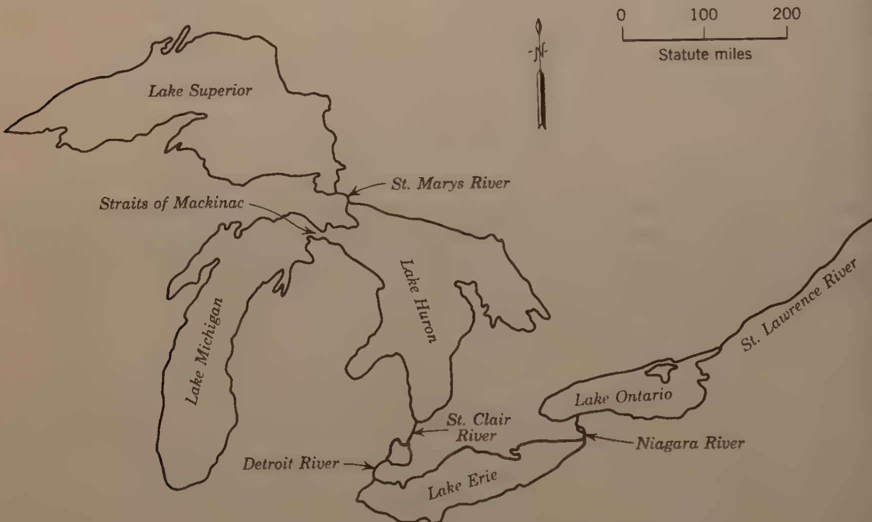

FIG. 1.—THE GREAT LAKES AND THEIR OUTFLOW RIVERS

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3 "Characteristics of Cup and Screw Current Meters," by B. F. Groat, Vol. 76, 1913, pp. 819-840.

4 "Chemi-Hydrometry and its Application to the Precise Testing of Hydro-Electric Generators," by B. F. Groat. Transactions, ASCE, Vol. 80, 1916, pp. 1231-1271.

5 "Stream Gauging," by William A. Liddell, first edition, McGraw-Hill, New York, 1927, pp. 136-150.

6 Discussion of paper by Edward C. Murphy, by Charles H. Miller, Transactions, ASCE, Vol. 47, 1902, p. 370.

7 Unpublished report of the U.S. Lake Survey, by Sherman Moore, file 3-1937, concerning tests of Price and Haskell current meters in St. Clair River in 1909.

8 Unpublished report of the U.S. Lake Survey, by W. S. Richmond, file 3-1973, concerning tests of Price and Haskell current meters in St. Clair River in 1910.

9 "Effect of Turbulence on the Registration of Current Meters," by D. L. Yarnell and F. A. Nagler, Transactions, ASCE, Vol. 95, 1931, pp. 766 795.

This paper presents first a brief description of the Lake Survey method of flow measurement to help the reader understand the comparison that was made, and then a description and analysis of the comparison.

UNITED STATES LAKE SURVEY METHOD OF FLOW MEASUREMENT

In selecting a vertical plane in the river, called a hydraulic section, past which the flow is to be measured, the flow characteristics looked for are lack of turbulence, uniform direction parallel to the banks, and velocities ranging between 1.5 and 5.0 ft per sec. Channel configurations sought are parallel banks, uniform cross sections upstream and downstream from the section, and relatively steep slopes at the water's edge. A section having these flow and channel characteristics can be selected tentatively for a given reach of the river by means of field reconnaissance and inspection of available hydrographic and topographic surveys. Before a section is finally selected, however, exploratory soundings and current velocity observations are made at intervals of 100 ft above and below the proposed section to verify the previous findings. The nature of the Great Lakes' connecting rivers is such that an acceptable hydraulic section usually can be found in any desired reach.

In the Lake Survey method, the hydraulic section is divided by vertical lines into a number (usually 10) of areas of nearly equal width, called panels. Current velocity measurements are made at a single specific location in each successive panel starting at one side or the other of the river. This location is usually at the horizontal mid-point of the panel and at a depth of 0.4 of the total depth at that point. A set of such velocity measurements for all panels in the hydraulic section, requiring two to three hours to obtain, is considered a single flow measurement, and the total flow computed from the data obtained is considered to be the average flow existing during the flow measurement.

The flow in cubic feet per second through each panel is computed by multiplying the measured velocity in feet per second by the area of the panel in square feet and by three coefficients which are called the directional coefficient, the vertical coefficient, and the horizontal coefficient. It is the purpose of the coefficients to convert the measured velocities into the mean velocities in each panel. The total river flow is computed by adding the panel flows.

The direction of flow through each panel is determined by transit tracking of floats, and the sine of the angle between the flow direction and hydraulic section is the directional coefficient. Normally, the directional coefficients are determined before the first flow measurements at a new hydraulic section are made, and they are not re-determined as long as the physical characteristics of the river in the vicinity of the hydraulic section remain the same.

A vertical coefficient for each panel is determined from a number of simultaneous observations of velocity at the measuring point of the panel at 0.4 depth and at each of the fractional depths 0.1, 0.2, 0.3, 0.5, 0.6, 0.7, 0.8, and 0.9. The average of the ratios of the velocities at the various depths to the simultaneous velocities at 0.4 depth is the vertical coefficient for the panel. Data for determination of vertical coefficients are usually obtained for a new hydraulic section before the first flow measurements are made, and the derived coefficients, like the directional coefficients, are not re-determined as long as the physical characteristics of the river in the vicinity of the hydraulic section remain the same. Under some circumstances, measurements of flow are made by metering the current velocities simultaneously at 0.2 and 0.8

depth in each panel instead of 0.4 depth. When this is done, vertical coefficients are not used; the flow through each panel is computed by multiplying the mean of the velocities simultaneously measured at 0.2 and 0.8 depth by the panel area and by the directional and horizontal coefficients only.

Horizontal coefficients are determined for each series of flow measurements at a given hydraulic section. Flow measurements are usually made in groups of about 20 over a period of about 10 days, and a new set of horizontal coefficients is derived for each such group of measurements from the 0.4 depth velocities observed during the measurements. The average of the velocities observed at 0.4 depth in each panel is plotted as an ordinate against the horizontal location of the measuring point as an abscissa. A smooth curve is drawn through the points so plotted for all the panels. This curve is considered to show the horizontal distribution of current velocities at 0.4 depth across the hydraulic section, and the horizontal coefficient for each panel is the ratio of the average ordinate of the curve over the width of the panel to the average of the observed 0.4 depth velocities.

Most Lake Survey flow measurements are made from a catamaran. Sometimes special conditions require that measurements be made from a bridge or a cableway. In measuring from a catamaran, velocity observations at each panel at 0.4 depth are made simultaneously with three current meters. The three meters are suspended on the line of the hydraulic section by cables, with a separation of 3 ft between adjacent meters.

COMPARATIVE TEST OF CUP AND SCREW METERS

A group of flow measurements was made by the Lake Survey in the fall of 1958 at the Stella Niagara hydraulic section in the lower Niagara River. The metering was done from a catamaran, simultaneous velocities for 5-min periods being observed with three current meters suspended at 0.4 depth in each panel. This provided a good opportunity to compare velocities obtained by different types of meters, and in all of the 20 flow measurements made over a period of 16 days, one cup and two screw meters were used simultaneously.

The cup meter used was the Price meter. Screw meters used were the Neyrpic, which is manufactured in France, and the Ott, which is manufactured in the Federal Republic of Germany. These three meters are shown in Fig. 2.

Three meters of each make were used for the measurements. Throughout a single flow measurement, the same three meters, one of each make, were operated in the same relative positions. In the successive measurements, a different meter of each make was used then in the preceding measurement and the three makes were placed in different relative positions so that about the same amount of data was obtained for each make in a given position.

Physical characteristics of the Stella Niagara hydraulic section are shown by the bottom profile in Fig. 3. The section is seen to be broad, shallow, and uniform. Also given in Fig. 3 is a curve showing the horizontal distribution of current velocities at 0.4 depth. In measuring the flow past the section, velocities were metered successively at the locations in Fig. 3 numbered 1 through 10 and labeled Panel Points.

ANALYSIS OF DATA

Table 1 shows the average velocities obtained at each metering location for each make of meter for the entire group of 20 flow measurements, and the

FIG. 2.—CURRENT METERS

average velocities for each make of meter for all metering locations and all measurements.

The horizontal distribution of velocity curve in Fig. 3 indicates that simultaneous velocities recorded by the three meters may not be the same because

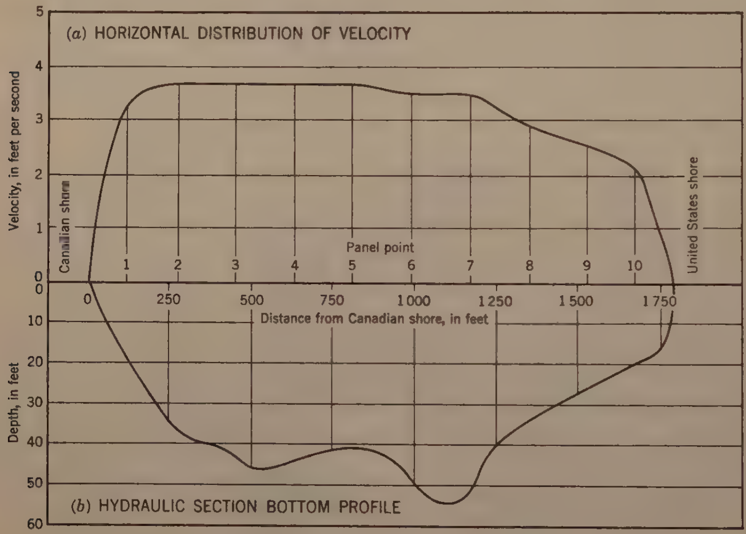

FIG. 3.—STELLA NIAGARA HYDRAULIC SECTION

of the slightly different positions of the meters on the line of the hydraulic section, particularly at panel points 1 and 10. Velocities at each panel point, compared with respect to position instead of type of meter, should reflect the

TABLE 1.—AVERAGE VELOCITIES IN fps OBTAINED FROM TWENTY 5-MIN METERINGS

Panel Point	Price	Neyrpic	Ott
1	3.288	3.290	3.294
2	3.676	3.676	3.682
3	3.700	3.702	3.703
4	3.718	3.719	3.718
5	3.679	3.684	3.698
6	3.540	3.542	3.544
7	3.474	3.474	3.480
8	2.898	2.902	2.911
9	2.526	2.544	2.536
10	2.188	2.216	2.192
All	3.269	3.275	3.276

shape of the horizontal distribution of velocity curve. Average velocities obtained from the twenty 5-min meterings at each of the three meter positions—left, center, and right—looking upstream at panel points 1 and 10 were computed, and the velocity differences between the left meter and center meter

and between the right meter and center meter were compared with corresponding differences indicated by the curve. The data are given in Table 2.

Although the data in Table 2 show that relative position has a definite effect on measured velocity, this phenomenon does not appreciably affect the data of Table 1 because of the interchange of meters between successive measurements.

TABLE 2.—COMPARISON OF VELOCITIES BASED ON METER POSITION

Panel Point	Meter Position	Mean Velocity Differences in Feet per Second Indicated Meter Position Minus Center Position Obtained from	
		Twenty 5 min meterings	Velocity curve
1	Left	+.031	+.03
	Right	-.034	-.03
10	Left	-.026	-.02
	Right	+.022	+.02

Comparisons of individual meterings of the three makes of meters, taken in pairs, were made. On Fig. 4 these comparisons are plotted at panel point 5 using the first and second minutes and third and fourth minutes of each 5-min metering as individual meterings for meter combinations Price-Ott, Price-Neyrpic, and Neyrpic-Ott. The points so plotted are in every case well centered around the line of equal velocity throughout the range of measured

TABLE 3. COMPARISON OF AVERAGE MEASURED FLOWS OBTAINED BY DIFFERENT METERS

Meter	Average Flow, in Cubic Feet per Second, for Measurements Computed Using Data from Indicated Meter Only	Percentage Variation of Flow Obtained by Single Meter from Flow Obtained by All Meters
Price	191,850	-0.13
Ott	192,420	+0.17
Neyrpic	192,440	+0.18
All meters	192,100	--

velocities. Some scatter of the points should be expected, but it is small, the points lying almost entirely within a 5% deviation from the equal velocity line.

A further comparison was made in terms of the average measured flows for all measurements using data from each make of meter exclusively, as shown in Table 3.

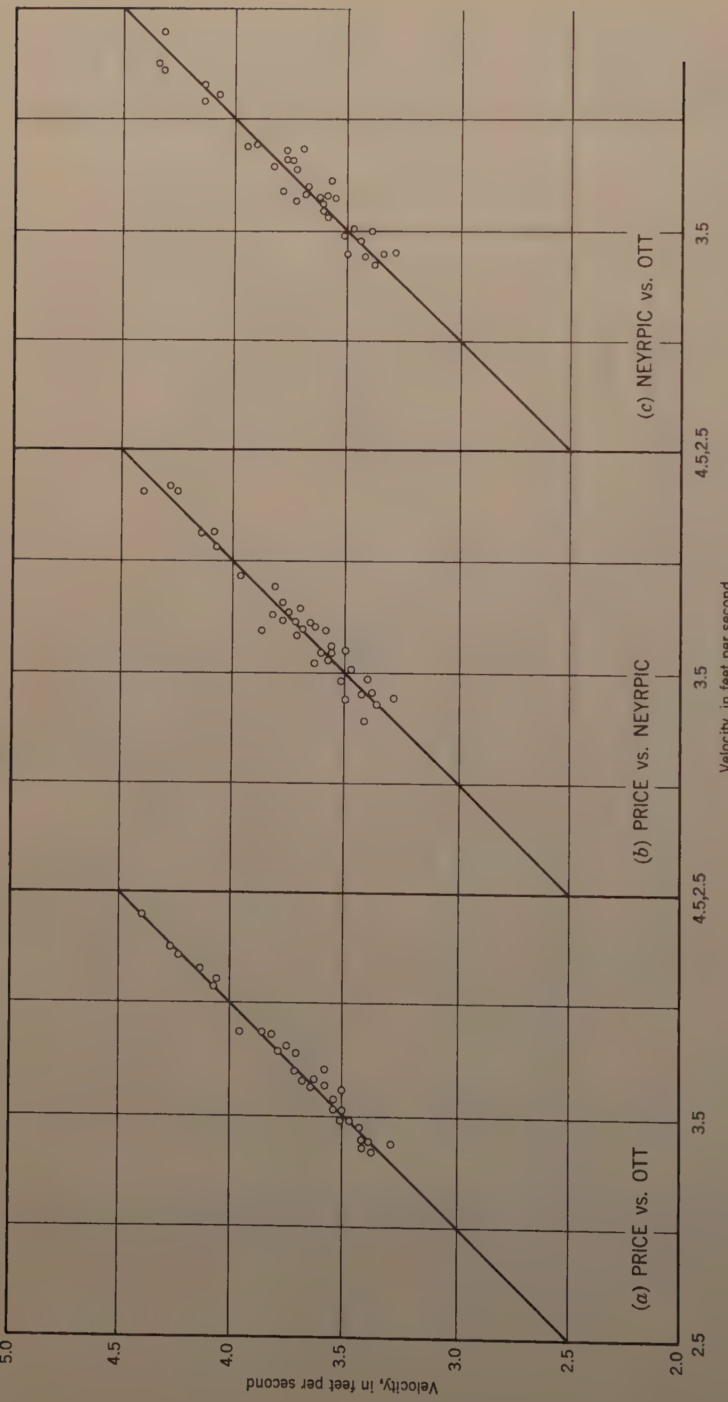

FIG. 4.—COMPARISON OF MEAN VELOCITIES MEASURED SIMULTANEOUSLY BY VARIOUS CURRENT METERS DURING 2-MIN PERIODS, STELLA NIAGARA SECTION, PANEL POINT 5

CONCLUSIONS

Based on the data presented herein, it is concluded that no greater accuracy can be achieved in Lake Survey flow measurements from a catamaran by using screw type meters instead of cup type current meters.

ACKNOWLEDGMENTS

The tests of current meters described herein, analysis of data obtained from the tests, and preparation of this report were accomplished as part of the Survey of Northern and Northwestern Lakes program of the United States Army Corps of Engineers by the U. S. Army Engineer District, Lake Survey. Presentation of this report to the American Society of Civil Engineers is by permission of the Chief of Engineers.

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EFFECTS OF FLOOD FLOW ON CHANNEL BOUNDARIES[a,b]

By D. A. Parsons[1]

SYNOPSIS

Some general and particular things of interest, learned in studies of stream bank stabilization methods in Western and Central New York State by the Agricultural Research Service, are given. The studies, for the most part, consist of attempts to relate observed effects of floods as evidenced by damages to stream bank revetment, to the channel geometry and the qualities of the flood flows.

INTRODUCTION

The Soil and Water Conservation Research Division, Agricultural Research Service, has been giving attention to stream bank stabilization problems in Western and Central New York State, particularly to those in the Buffalo River watershed.[2] Although the studies have far to go to meet the objectives of a good understanding of flood flows and economical means for

Note.—Discussion open until September 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

[a] The work of the Agric. Research Service, U.S. Dept. of Agric. in New York, is in cooperation with the Cornell Univ. Agric. Experiment Stations and the State of New York Conservation Dept.

[b] Presented at the May 1959 ASCE Convention in Cleveland, Ohio.

[1] Hydr. Engr., Agric. Research Service, Soil and Water Conservation Research Div., Watershed Tech. Research Branch, East Aurora, N.Y.

[2] "Streambank Stabilization," by Leon F. Silberberger. Agricultural Engineering, Vol. 40, No. 4, April, 1959, pp. 214-217.

stabilization of channels, some general and some particular things have been learned that are of interest. Most of the observations have to do with patterns of damage to stabilized channels. Of interest, too, may be the measurements, the thinking, and the methods by which attempts are being made to relate the damages, or instability, of the channel bank to the flood flows that occurred.

From the standpoint of evaluation and development of streambank-protection methods, it seems proper to separate the elements involved: the stream and its qualities that determine the potential for bank erosion, and the boundaries and their qualities of resistance to the attack of the stream. These two things are not completely independent, because in natural streams the flows create the channel and conversely the channel and flood plain geometry greatly influence the flow qualities. Yet, how else can a rational design for channel stability be achieved than by matching the resistance of the boundary against the intensity of the attack of the flow? This requires quantitative measures of both.

Elements of the boundary are moved (eroded) because of the forces exerted upon them. It is common knowledge that these forces are primarily dependent on the velocity of the fluid in the immediate vicinity of the protruding portions of the elements. It seems then that to evaluate the potential of the flood flows to erode the boundary or destroy the bank lining, it is necessary to measure the water speeds near the boundary. This, of course, can be done, but the question arises, how close should the measurement be made? The speed of the water far removed from the boundary is not of direct concern.

Perhaps the fluid shear stress at the boundary is a preferable measure. Its dimension is force per unit area and, although shear stress does not seem to be very descriptive of the actual forces that act upon the elements of the channel boundary, its dimension seems proper. But here, too, there are difficulties. It has not yet been found easy to measure the boundary stress from spot to spot. It seems, too, that the boundary drag is a good measure of the intensity of attack of the stream only to the extent that it reflects the forces that would act upon a protruding element.

Briefly, the work in New York is directed, among other things, toward the determination of the potential of flood flows to destroy the channel boundaries from spot to spot for many geometrical situations of channel alinement and bank height. Quantitative measures of some type are required. These may be the boundary shear stress, but thus far, reliance has been on comparisons of the flood-flow qualities of mean velocity head and mean tractive force for the channel with the measured sizes of riprap stones for the condition of incipient movement. The use of the measurable, mean qualities of the channel flow is not only helpful in investigations; it is quite essential to application in design of any relationships that are developed.

Coarse materials transported by the flood flows are factors in the process. They may be capable of materially altering the flood flows and flood stages, as with ice jams, trees blocking bridge openings, etc., but, for the most part, their role in the bank-erosion process appears to be principally as agents or helpers of the flow in the attack on the bank.

In the case of ice, it is quite usual to find the motion of the pack ice, following an ice jam, confined solely to the channel with vertical shear planes at about the toe of the bank. The ice along the bank thus acts as a buffer for the bank against the potentially destructive floes within the channel. However, it has been observed that this buffer ice is often insufficient to withstand the following onslaught of the stream in those places along the boundary normally subject to the heaviest action of the flood flows.

Heavy objects like rooted trees, stumps, and boulders may drag or catch on the channel bottom, causing higher water speeds along the banks, especially for the smaller streams, than would normally prevail. The materials carried higher in the flow appear to be true agents of the flow, augmenting the ability of the water in its action.

The manner of the erosive action of transported materials is varied. The processes include impact of moving objects with the bank lining, an abrasion, and sometimes a lodgment against a protruding element of the lining, thereby increasing the force on the element tending to move it out of place. The effectiveness of these processes is dependent on the magnitudes of the water speeds in the immediate vicinity of the boundary and the transported object.

STREAM CONDITIONS

The usual stream conditions in Buffalo Creek, N. Y., range from a width of about 65 ft, slope 0.006, probable annual peak flow 2,500 cfs above Java Village to a width of about 120 ft, slope 0.0025, and probable annual peak flow 8,000 cfs at the lower end. The bed material is sand, gravel, and cobbles, predominantly gravel. The stream has a recent history of degradation in some places, particularly in the upper reaches, and flows on nearly smooth bed rock in some places, especially toward its lower end. Floods are numerous, occurring mainly in the winter and early spring. Ice conditions are sometimes severe in the winter floods.

GENERAL OBSERVATIONS

Most streams are continuously changing their channels by aggradation or degradation of the beds and by the building up and tearing down of the banks. This is inherent in natural streams. Therefore, prior to initiation of any major channel work or change in flow and bed material transport regimen, there is need to determine the current trends of channel change, especially any progressive changes in channel-bed elevation. It is also essential to predetermine the effects of the installed measures on future stream-bed elevations, in addition to the lateral migrations of the channel, before the plan for work can be truly called a design for improvements. Both benefits and damages are partly dependent on the future elevation of the channel bed; so also, is the exact design of most measures for stream-bank protection. Some highly undesirable channel conditions have been observed that were due to the neglect of one or both of these work-prerequisites.

It should not be necessary to prove the obvious, that is, that the potential of flood flow to erode the channel boundary varies from spot to spot. It is desirable though to restate some elemental facts that tend to show this. They cannot be over-emphasized, either from the standpoint of channel investigations and evaluations of protective measures, or from the standpoint of actual bank-stabilization work. They have to do with meandering channels. Pending the time when studies of secondary currents, helical flows and variations in tractive forces over the wetted portions of the channel provide the means for determining exceptions to these simple rules, they may not be ignored with impunity.

C. R. Allen presented[3] a convincing description of the mystic tendency fo the meander pattern to move down valley. He said, in part:

> "In 1892 the writer meandered forty miles of the Des Moines River and plotted the same in relation to the U. S. Survey made in 1847. I found, which I presume is not new, but a well-known law, that whenever the reaches ran parallel with the general course of the river there was little change at the shore line by erosion; that wherever the reaches ran transversely to the general course. . . the banks have changed by erosion downstream."

This tendency for the down-valley migration of the meander pattern ha been observed many times and in many places since then, and when considere with the erosion of the outside bank in bends, obvious to all, it must be con cluded that the potential of the stream to erode its banks not only varies fro one spot to another, but is persistently high in some places. For stabilit these places along the bank must be provided with a more resistant materi than usually occurs in nature. It is just as important, too, to realize that ther is no need for protective work along the opposite shore. The opposite shor tends to build up by deposition.

The potential of the flood waters to erode the bank also varies up and dow the bank slope, being less in the shallow water near the water's edge tha deeper in the flow. A better knowledge of its variation from the toe of the ban to the water surface at stream edge would be of aid in revetment design. Muc of the present knowledge of this variation for straight channels has been sum marized by E. W. Lane.[4]

The intensity of attack of the flood waters at a spot along the boundar generally increases with the rarity of the flood, and varies with time fo steady flows because of the turbulence and large-scale eddies of the floo waters. The chief concern in design is therefore with that greatest flood flo associated with the allowable risk of exceedance within the design life of th stabilization work. However, lesser flows may produce more severe condi tions at some places along the bank and should be considered.

It has been found that the pertinent mean-flow qualities of the flood water within and adjacent to the channel are somewhat different for the larger flood than might be deduced by extrapolation of values from smaller flows com pletely confined to the channel. The topographical features of the valley floo and walls that tend to restrict the flow, including highway fills and artifici structures, become important. Sharp channel bends, immediately downstrea from the bank areas under consideration, also become important in the large floods. High energy losses in the larger flows through these bends tend to re duce the energy gradient immediately upstream.

Some particular and generally localized effects of floods have been ob served. For the most part, these observed effects are the complete or parti failure of established protective measures. These spots are almost entire

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3 "Protection of River Banks at Ottumwa, Iowa," by C. R. Allen. Iowa Civil Engi neers and Surveyors Society, Proceedings of the Seventh Annual Convention, January 1895, pp. 39-46.

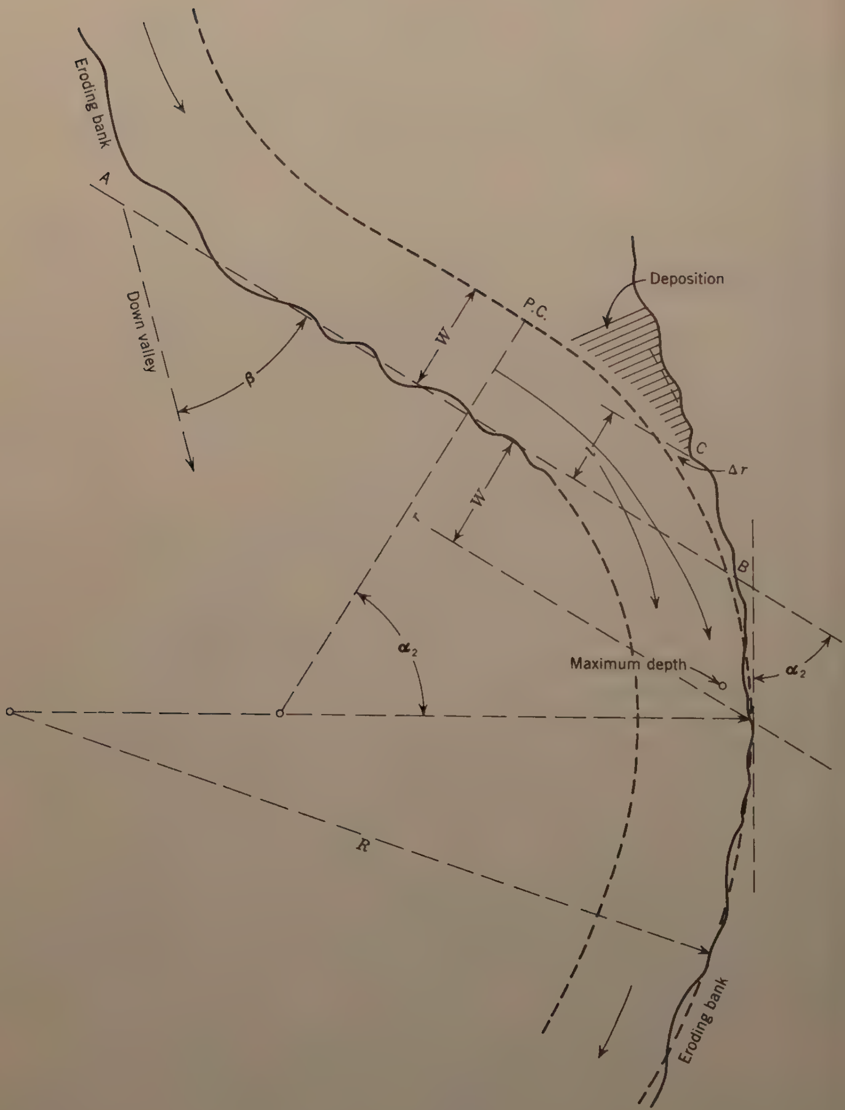
4 "Design of Stable Channels," by E. W. Lane. Transactions, ASCE, Vol. 120, 1955 p. 1234.

confined to the down-valley bank and in places where the boundary currents are strong. The observations indicate the following:

1. There is a portion of the bend wherein the water is deepest and the attack of the stream greater than normal.
2. Great difficulty is experienced in holding the protective works along the down-valley bank in those places where the banks are low. High-speed water leaving the channel creates a condition that is more severe than usual. It has been found helpful in some places to build the bank up above the flood stage before revetment.
3. Out-of-bank flood waters, returning in a concentrated fashion over the bank to the main channel, are often able to damage the protective works if it is not somehow re-enforced.
4. Inadequate distance between revetted banks, constituting a restriction in the channel width, increases the severity of the action of the flood flows on both banks. In a meandering channel this could ordinarily occur only near the downstream end of the revetment on one bank and the beginning of the revetment on the opposite bank.
5. Protrusions into the flow are subject to high forces.
6. Irregularities in the alinement of the bank induce severe action by the stream on those exposed portions.
7. Failure of the bank-stabilization work is almost always in the nature of an erosion, that is, a little bit at a time. The damage to, or loss of, one small element of the lining may occur almost instantaneously; but several units of time, a whole flood, or a succession of floods may be required to disintegrate the lining to the extent that the bank stability is lost. In fact, this process of disintegration is tacitly recognized in one method of stabilization. Periodic feeding of semi-resistant materials to an eroding area can effectively control the migration of the shore or bank.
8. An effect opposite to the preceding is the continued progression and build up in the down-valley direction (Fig. 1) of the shore after stabilization of the opposite eroding bank of a rapidly migrating stream. A particular point of interest in this is the extent in plan of this deposition at the beginning of bends.
9. The fluctuating boundary currents with associated pressure variations, aided by surges, waves, seiches, and sometimes ground-water flow to the stream, tend to cause loss of the finer bank materials from beneath the protective veneer along the face of the bank. Failure in this fashion of the supporting base of the lining has frequently caused complete failure of otherwise excellent revetment.

PATTERNS OF FLOOD EFFECTS IN BENDS

Fig. 1 illustrates a typical stream-channel situation and a manner of study of stream behavior. A reference line, A-B, is drawn or sighted along the eroding down-valley bank. It is not surprising to find that the alinement of this bank is the key to the bed geometry and to the erosive forces in the channel bend immediately downstream. Instability of the bank, represented by the line A-B, with consequent down-valley migration, immediately alters the situation in the bend below. The design alinement and stabilization of this bank should therefore precede those of the one downstream.



Eroding bank
A
Down valley
β
W
P.C.
Deposition
C
Δr
W
B
α2
Maximum depth
α2
R
Eroding bank

FIG. 1

Flood waters moving along parallel to line A-B tend to continue in the same direction, but as they approach the down-stream bend a force is exerted upon the flow, tending to change its direction. The slowly moving water near the bed and the banks is more readily turned. The fast water near the surface, relatively free of suspended bed materials, crosses over to be in close proximity with the outside bank, somewhat as indicated by the arrows.

Some of the observations of partial or complete revetment failure in bends are portrayed in Fig. 2. Also shown are locations of several deposition points in the upstream parts of bends following bank stabilization work, and the locations of deep points in bends of unstabilized channels. Horizontal distances have been divided by stream width to give a dimensionless representation. It is not expected to find true similitude in nature; yet, considering the order of magnitude of the dispersion of the phenomena with the possible error inherent in the assumption, along with the great utility in dimensionless representation, similarity of streams of different size is assumed.

Even stream width is a quality of large dispersion, especially for migrating streams. For rapidly migrating streams there is no well defined up-valley bank, and one needs to search for sites to measure and average.

The position of the deposition point C in Fig. 1 is an important consideration since it is logically associated with the beginning point of need for a rugged type of revetment. A rapidly migrating stream would leave the bank in this area in a raw condition. This superficially indicates the need for strong revetment much further upstream than is truly the case. Common misjudgments in stream-bank-protection work are to revet this bank too far upstream and fail to go far enough downstream into the bend on the opposite side. This is a result of the otherwise good, simple rule of reveting the bank that is eroding.

The true significance of the observed location of the downstream deposition limit at the beginning of bends has not been established. Referring to Fig. 1, the condition seems to be that the stream flows freely in a curved path as it passes from one restraining bank to the other on the opposite side. Deposition progresses down valley to conform with the stream path. Its point of ending, therefore, presumably would depend upon the exact location of the previously eroding bank in addition to the curved alinement that the stream adopts. The extent of the revetment on the opposite side is also a factor. The distance designated Δr between the point C and arc with radius r was close to and averaged 0.04 W for 10 out of 12 measurements.

The inference of the plotted representation of damages in bends is that the erosive forces of the stream begin to become severe at the point B, reach a maximum in severity at a point on the order of one stream width away from the reference line, depending somewhat on the bend radius and approach conditions, and then moderate further along in the bend. The variation from the average picture is considerable. There are several reasons why this should be so. The tacit assumption of uniform revetment strength throughout each of the bends is undoubtedly untrue. The approach conditions varied from bend to bend. Also, the adopted position of the reference line may not have been the effective one in every case.

The picture of damages to protective works in Fig. 2 may be of aid by indicating, in a rough way, the places in bends that are particularly difficult to protect. But it leaves much to be desired, if one is faced with the need to specify precisely the protective works to be installed.

REQUISITE SIZES OF RIPRAP STONES

Whereas data are not adequate to indicate with confidence the strength of the required revetments, knowledge is not completely lacking. The question is whether to wait for additional measurements to augment the very limited information that is available, or use it as a basis for a first guess. Since the required data are difficultly and slowly obtained, and since the need for quantitative estimates are current, the guess will be made. The explanation of the estimate will also show the methods that seem to be required to derive the needed knowledge. Actually, in making these studies it seems necessary to set up trial or tentative relationships such as Eq. 3 (to be presented subsequently) and make some preliminary assumptions in regard to the pertinent flow parameters and relative resistances. If the observations do not conform with the assumptions, new schemes are tried.

There is need first to explain the origin of coordinates of Fig. 2. Each of the deep points and damage areas were located on a plan of the stream channel, similar to that of Fig. 1. A line was then drawn tangent to the eroding bank at a point one stream width below the reference line. The angle between this tangent line and the reference line is α_2. It was then assumed that the path of the stream as it entered the bend was that of a circular curve such that

$$\frac{r}{w} = \frac{2}{1-\cos \alpha_2} \qquad (1)$$

The origin of coordinates for Fig. 2 corresponds to the point on Fig. 1 marked P.C. If the tangent line is extended to intersect a line one stream width up valley from A-B, the distance from the intersection to the point P.C., the origin of coordinates, is $(r/w) \tan (\alpha_2/2)$. The Y and Y/W direction in Fig. 2 is the same as that of the reference line A-B in Fig. 1. There undoubtedly is a better way of representing the observations. The angle α_2 may not be the best angle to select and a curve other than circular may be more representative. However, a better manner of representation of the bend geometry should not be greatly different for stream conditions like those observed.

The straight line,

$$\frac{Y}{W} = 7\left[\frac{X}{W} - \frac{4}{3}\right] \qquad (2)$$

in Fig. 2 is drawn to represent the positions of maximum intensity of attack in bends. The requisite strength of revetment along this line, as determined from field experience, is[c]

$$P'' = \left[4 + \frac{50}{2 + (Y/W)_p}\right] Z \qquad (3)$$

or

$$P = \left[\frac{1}{3} + \frac{4.2}{2 + (Y/W)_p}\right] Z \qquad (4)$$

[c] Eq. 3 is a revision of a simpler one given at the May, 1959 Cleveland, Ohio meeting of the ASCE and is believed to be much better for the very large and very small values of $(Y/W)_p$.

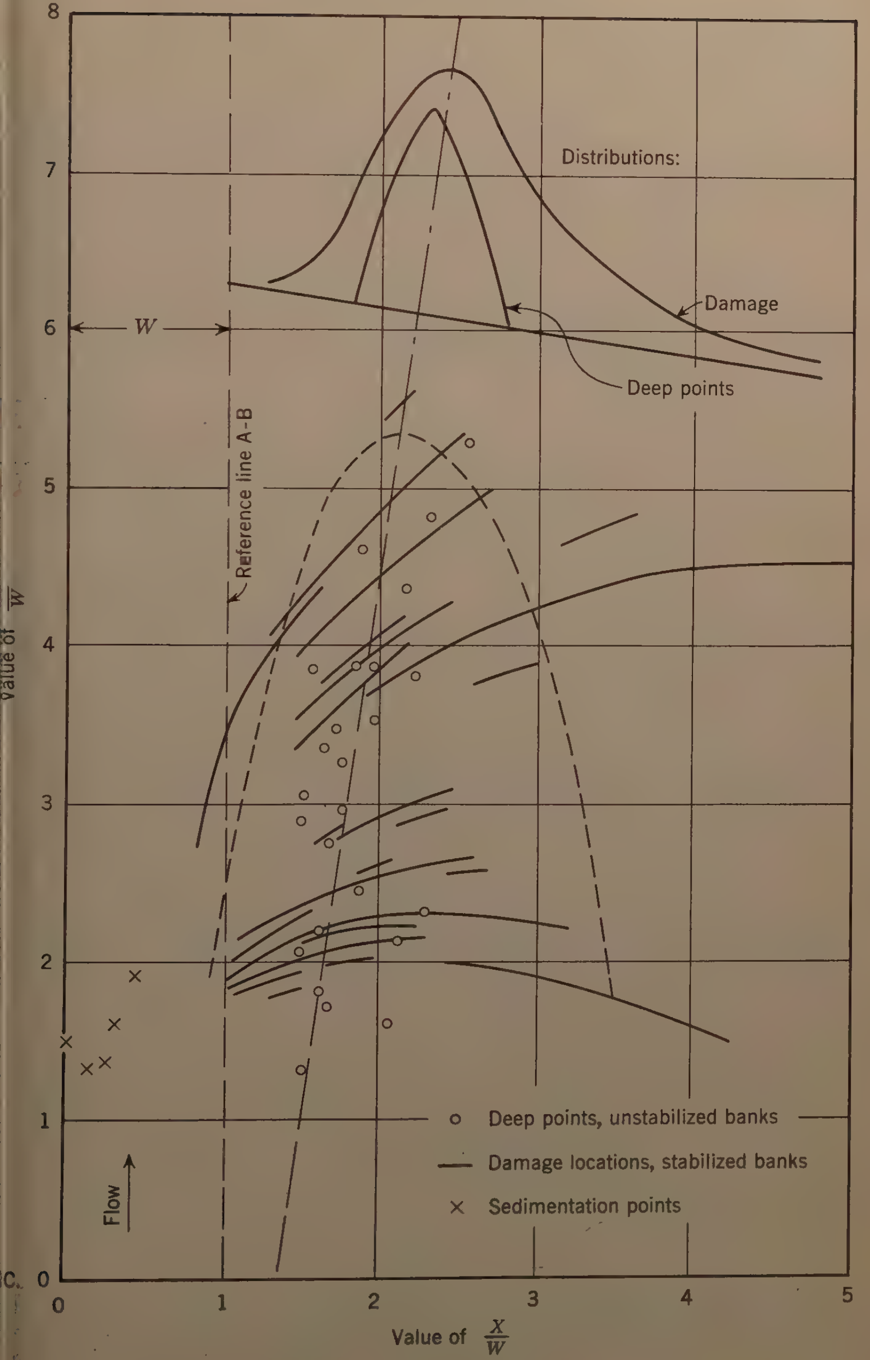

Distributions:
Damage
Deep points
W
Reference line A-B
Flow
Deep points, unstabilized banks
Damage locations, stabilized banks
Sedimentation points
Value of $\frac{X}{W}$
0
1
2
3
4
5
6
7
8

FIG. 2

In Eqs. 3 and 4 P'' and P are the minimum requisite strength, expressed as an equivalent median riprap stone diameter in inches or feet respectively, on a 1 or 2 bank slope, placed with only moderate care, and subjected to stream conditions as found in western and central New York State; the subscript p indicates the coordinate to the point of maximum attack in bends; and Z is a flow parameter descriptive of the within-channel flood flow, either the velocity head ($V^2/2\,g$) in feet or the mean tractive force (T_o) in pounds per square foot. It happens to be that for the damaging flood flows that have been measured in Buffalo Creek, the average values of these two parameters are about equal. However, this is not a general condition, and use of the velocity head is favored at the present time. Lines of equal required strength are presumed to be positioned about the line of the maximum, somewhat as is the envelope curve for the bulk of the damage data. Fig. 3 shows values of (r/w), $(x/w)_p$, $(Y/W)_p$ and (P/Z) as they vary with α_2.

Eqs. 3 and 4 are strictly empirical and were chosen, because of their simplicity, from many others, that might have been used to represent the limited information. The constant, 4 in Eq. 3 was chosen simply as a reasonable figure, but it gives results quite close, using the velocity head for Z, to the values for gravel as given[5] by S. Fortier and F. C. Scobey for straight channels. The line marked P/Z in Fig. 3 may be used in lieu of Eq. 4.

Most of the damage data were for quarried stone revetment with a specified size of approximately D = 34 p where D is the size in inches and p is the proportion by weight that is smaller. Thus D_{50} = P'' = 17 in. The flood flows that caused the damage were measured in only a part of the cases. This was done by means of post-flood surveys of peak water-surface elevations and channel cross sections from which peak discharges, mean velocities, slopes and mean boundary stresses were obtained. The values of T_o and $V^2/2\,g$ thus determined were, of course, variable, but averaged about 3/2. The envelope curve for damages crosses the straight line at $(Y/W)_p$ = 5.3. Using these values, Eq. 3 gives 16 in. for the median stone size.

A few other particular cases of the same and of different bank linings were studied in the same fashion with reasonably good agreement. In one case where $(Y/W)_p$ = 8.4, there was a vegetative lining of a combination of excellent 3-yr growth of grass and woody vegetation including basket willow. Its resistance was apparently exceeded for about 40 ft (0.6 W) along the outside of a long-radius bend on Bennettsville Creek, N. Y. The stream survey and testimony of the land owner resulted in an estimate of 1 lb per sq ft for the T and 0.55 for $V^2/2\,g$ for the flood that caused the damage. Eq. 3 yields 9 in. and 5 in. respectively for the equivalent median stone size as a measure of the maximum intensity of attack. For dumped, loose stone, these values would be increased to 11 in. and 6 in., respectively. This compares with a maximum of 6 in. for the resistance of a good stand of long, green bermuda grass as determined from the data[6] of W. O. Ree and V. J. Palmer and the results of flume tests for critical tractive force.

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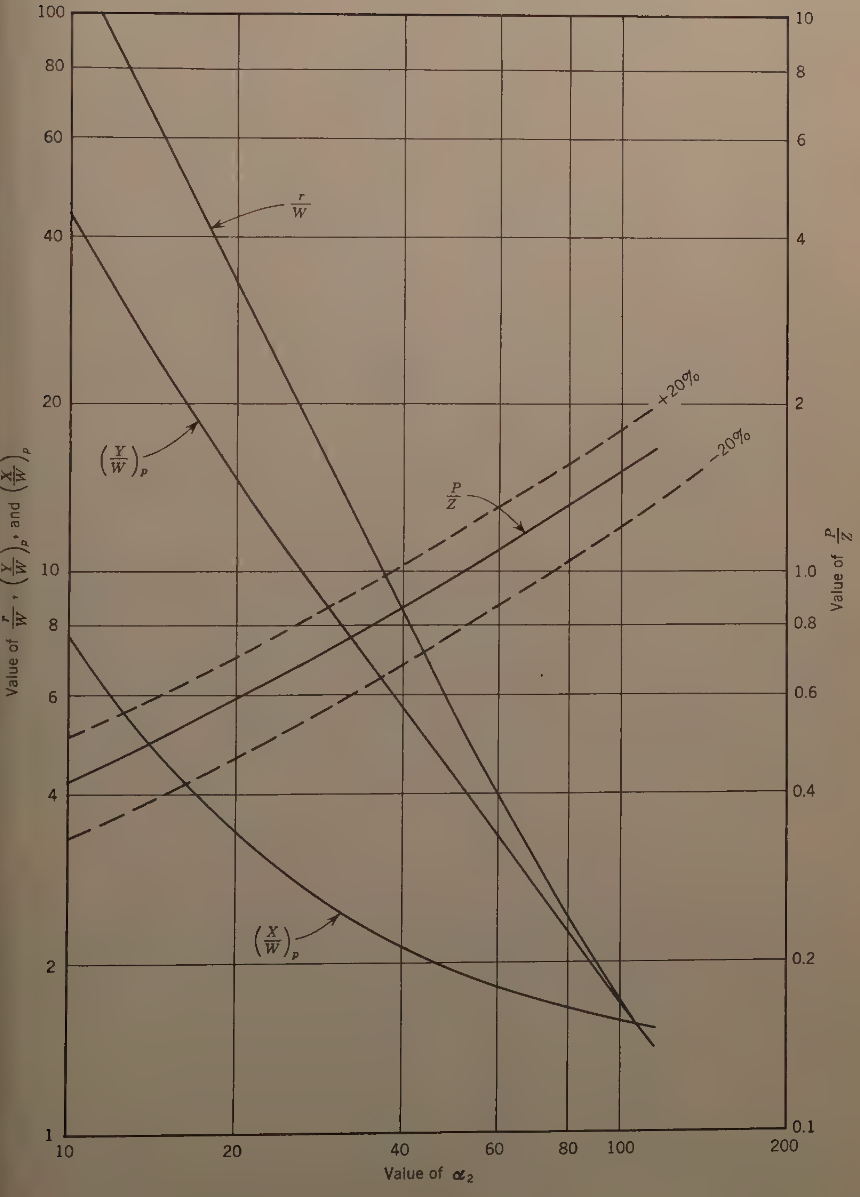
[5] "Permissible Canal Velocities," by S. Fortier and F. C. Scobey. Transactions ASCE, Vol. 89, 1926, p. 940.

[6] "Flow of Water in Channels Protected by Vegetative Linings," by W. O. Ree and V. J. Palmer. U.S. Department of Agriculture, Technical Bulletin No. 967, February 1949.



Value of $\frac{r}{W}$, $\left(\frac{Y}{W}\right)_p$, and $\left(\frac{X}{W}\right)_p$
Value of $\frac{P}{Z}$
Value of α_2
$\frac{r}{W}$
$\left(\frac{Y}{W}\right)_p$
$\left(\frac{X}{W}\right)_p$
$\frac{P}{Z}$
+20%
−20%
100
80
60
40
20
10
8
6
4
2
1
10
8
6
4
2
1.0
0.8
0.6
0.4
0.2
0.1
10
20
40
60
80
100
200

FIG. 3

At the other end of the line it has been observed that stones approaching 1 ton in weight (about 35 in. equivalent diameter) have been moved under extreme conditions along Buffalo Creek, N. Y., when partially isolated by loss of the finer stones or laid in an exposed manner. In a sharp bend with, say, $(Y/W)_p = 1.5$ and T_o or $V^2/2\,g = 1.8$ representing an extreme condition in Buffalo Creek, Eq. 3 for median stones gives 33 in. There are no measurements to check on the validity of this estimate of the median size required for stability, but from experience with the creek it seems to be a reasonable value.

A bend in Page Brook, N. Y., on the Verner Lewis farm, revetted with dumped riprap stones of 7-1/2 in. median diameter, lost a few stones in a length of about 0.2 W (7 ft). The $(Y/W)_p$ as defined was 4.54. Estimated values, based on the stream survey, for T_o and $V^2/2\,g$ were 1.00 and 0.49, respectively. Eq. 3 yields 11.7 in. using T_o and 5.7 in. using the velocity head. Here again the computed sizes are for stones placed with moderate care. One of the assumptions currently being made is that dumped stones need to be about 20% larger, and stones placed with much care to eliminate protrusions from the face of the blanket about 20% smaller. Then the estimates for dumped stones in this bend in Page Brook for the flood experienced is 14 in. and 6.8 in., respectively. The bank slope in this bend was 1 on $1\frac{2}{3}$, a little steeper than is applicable with Eq. 3.

A riprapped bend on the E. Foss farm along Buffalo Creek experienced two almost identical floods of about 25-yr expectancy. Post-flood surveys indicated flows of about 10,000 cfs and 0.5 ft per sec mean velocity. The $(Y/W)_p$ is 2.96, $T_o = 1.5$ and $V^2/2\,g = 1.44$. The specified median stone size was 17 in. Eq. 3 yields 21 in. and 20.3 in., respectively. Since this was contract work and much care was used in placement of the rock, the requisite size would be somewhat less. Values 20% less would be 16.8 in. and 16.3 in. The particular interest in this bend arose because the intensity of flood-water attack and the revetment strength appeared to be almost perfectly matched. Some of the smaller stones on the face of the bank were washed away and several of the large ones had been shifted slightly, but after two major flood experiences, the revetment was still in good condition.

SOME ADDITIONAL ASPECTS OF REVETMENT DESIGN

The stability of protective linings appears to be considerably affected by the extent to which the elemental parts jut into the flow. Although no quantitative measurements were made, there is evidence to suggest this. The helter-skelter arrangement of the stones of dumped riprap have appeared to be less effective than when the lining was placed by crane and clam shell with a moderate attempt at elimination of irregularities on the face of the blanket. Work done with a considerable effort to fit the stones together, chink the holes, and remove all large protrusions has generally stood up still better. Impressive, too, in this respect, is the comparative stability of closely fitted experimental concrete revetment blocks. These blocks were 24 in. by 16 in. by 4 in., each having twenty four 2 in. by 2 in. by 4 in. holes and weighing 83 lb. Out of about six hundred on the bank of a bend, in a spot that the formula suggests the need for 26 in. stones (800 lb) two blocks at exposed corners were lost to the floods; whereas a considerable quantity of stone of 17 in. specified diame-

r was lost, both upstream and downstream of the experiment. The relatively od showing of the concrete blocks in this respect is believed due to the fact at they provided little opportunity for the water and transported materials impinge upon their upstream edges.

Irregularities in stabilized bank alinement that appear to induce a more in- nse attack by the stream are of several types. A poorly defined notion exists at the stream tends to follow a smoothly curving path of decreasing curva- re throughout a bend, and that banks encroaching on this path are more lnerable than normal. Specifically, in several cases where the curvature of e downstream portion of a bend is greater than that of the apparent free-flow rvature of the stream as it enters the bend, the lower portion of the bank in e bend is not only subjected to heavy action, but its presence seems to re- eve somewhat the intensity of the expected action immediately upstream. esumably, a large loss of energy in the downstream portion during large ods causes a reduction in the energy gradient upstream, with consequent duction in water speeds.

The downstream shoulder of the bank that is created by a depression in e revetment for a stream crossing for farm vehicles or in the mouth of a nall tributary is particularly vulnerable.

In the one case observed and shown on Fig. 2 where the upstream end of the vetment in a bend appears to have been extended out into the path of the ream, severe action was experienced. If the downstream end of revetment es not curve down valley, somewhat in conformance with the path of the w, it will in effect be a protrusion into the flow and need to be stronger than nerwise.

The loss of fine materials from beneath a portion of the revetment has en experienced with the experimental concrete blocks. This was not evident til a flood eroded away the adjoining stone riprap revetment and the bank aterial along the edges of the experimental mat. Fine material from be- ath the blocks was lost adjacent to the exposed edges. Little, if any, was st from the small holes in the blocks. The mat appears to be stable because ich of it was laid on a layer of sand, gravel, and cobbles. The coarser ma- rial of the foundation was not moved out by the flood. The experience with s one flood, however, appears to demonstrate one mode of failure of mat pes of lining.

BOUNDARY SHEAR STRESS EVALUATIONS

Measurements of the depths of scour of materials of known size and density m some of the 2 in. by 2 in. holes in the experimental revetment have been de. This was an attempt to delineate the variation in the intensity of action the flood flows against the bank. Estimates of the boundary stresses have en made, but the technique of analysis needs verification. Also, the scour- pth data were very erratic for reasons only partly understood. For these asons, they will not be reported here, except to say the depths of scour were finitely greater in the larger floods; the scour was greater deep in the flow n where the water was shallow, and the mean scour differed for different ctions along the bank. Good results are anticipated along this line from the rrent studies of flows in laboratory channels at the Hydrodynamics Labora- y, Massachusetts Institute of Technology, Cambridge, Mass. A comparison

of the boundary shear stress variation throughout a 60° bend as reported[7] b R. E. Nece, C. A. Givler and P. A. Drinker with the pattern of flood effects i natural stream channels is encouraging.

MAINTENANCE CONSIDERATION

A discussion of stream bank stabilization work and flood effects would b incomplete without some mention of maintenance. A provision for some kin of maintenance effort is an absolute must for stream bank stabilization wor to remain effective. There are so many uncertainties about design and chanc happenings that may affect the stability that constant watching and prompt ac tion are a necessity.

Actually, the expected quality and timeliness of the maintenance effort is factor in design. One farmer was observed to be doing an excellent job c maintaining stable banks of a small stream. The riprap stones that he use were from his own fields and were too small for the toughest spots in th bends. He made the stone blanket thicker in these spots. If a large floo washed away some of the material he would promptly haul in another loac The strength, extent, and quality of initial stabilization work should be in creased as the level of expected attention to maintenance goes down.

SOME OTHER FACTORS INVOLVED

The principal direction of the work in New York on this problem has bee toward the flow element of the process, because it seems to be the most neg lected and toughest portion. Needless to say, there is much yet to be done For the most port, too, the emphasis has been on complete stabilizatior rather than on methods for reduction of stream-bank erosion to tolerabl rates, although an understanding of the flow capabilities should help in this The various methods for complete stabilization have hardly been touche upon. Cost is paramount in this field, and each method has peculiar problem of its own. The highly important questions of changes in streambed elevatic have been given only minor attention.

The use of vegetation in streambank stabilization has been given muc study in the Buffalo Creek work. Real progress has been made in this fiel The observations and the conclusions in regard to the place of vegetation hav been ably expressed[8] by Harry Porter and Leon Silberberger, M. ASCE.

ACKNOWLEDGMENTS

The material aid of Soil Conservation Service personnel in New York Stat and the Buffalo Creek Flood Prevention Project in particular is gratefull acknowledged. Most of the observations could not have been made without it

[7] "Measurement of Boundary Shear Stress in an Open Channel Curve with a Surfac Pitot Tube," by R. E. Nece, C. A. Givler and P. A. Drinker. Massachusetts Institute Technology Hydrodynamics Laboratory, Technical Note No. 6, August, 1959.

[8] "The Use of Vegetation in Streambank Stabilization," by Harry L. Porter and Lec F. Silberberger. Soil Science Society of America, 1958 meeting. To be published in th Journal of Soil and Water Conservation.

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FRICTION LOSSES IN LINES WITH SERVICE CONNECTIONS

By David L. Muss,[1] M. ASCE

SYNOPSIS

The determination of friction losses in water mains is complicated by ser-ice connections along the line which result in different flows in each section f the pipe. One method of determining these losses[2] has been to assume that e amount actually withdrawn along the line is withdrawn as a single quantity t the end of the line. This assumption may result in the computation of fric-on losses which may be as much as 2.85 times the correct value, depending n the number of connections and the ratio of the quantity withdrawn to the total let flow. European practice has been to assume that 45% of the amount with-rawn is removed at the beginning of the section and 55% at its end.[3,4] Although is approximation results in a small error (-5.7% to +3.5%) for a large number f connections, the value of the friction loss computed for a small number of onnections may be as little as 35% of the correct value.

Two alternate methods of approach are proposed. Both are simple and ac-urate. The first method, applicable only to dead-end systems, or other situa-ons where the flows are known, involves the selection of either a correction ctor, to be applied to the flow entering the line, or to the friction losses com-ted on the basis of the inlet flow or to the length of pipe. Factors vary from 568 to 1.000 for equivalent flow and from 0.351 to 1.000 for friction loss or

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Note.—Discussion open until September 1, 1960. To extend the closing date one onth, a written request must be filed with the Executive Secretary, ASCE. This paper part of the copyrighted Journal of the Hydraulics Division, Proceedings of the merican Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

[1] Cons. Engr., Asst. Prof. of Civ. Engrg., City College of New York, N.Y.

[2] "Water Supply and Purification," by W. A. Hardenbergh, Internat. Textbook Co. ranton, Pa., 1952.

[3] "Idralica," by Umberto Puppini, Nicola Zanichelli, Bologna, Italy, 1947.

[4] "Vodosnabjenie," by N. N. Abramov, N. N. Geniev and V. I. Paplov (Water Supply gineering), Moscow, Russia, 1958.

equivalent length. This method is precise and there is no error in its applica-tion.

The second method is applicable to either the dead end or loop system, an the flows need not be known in advance. A portion of the flow, of the amou withdrawn, is assumed to be withdrawn at the beginning of the line and th balance from the end of the line. This portion varies from 0 to 44% dependir on the number of connections. This method is approximate, but the range error is small and is limited to from -3.5% to +5.5%.

INTRODUCTION

The determination of friction losses in water mains is complicated by th service connections which remove water from the main along its path. Th results in a different value of flow from each service connection. A conserva-tive simplification of this problem is to assume that the quantity removed alor a given length of main is instead removed as a single quantity at the end of th main as shown in Fig. 1.

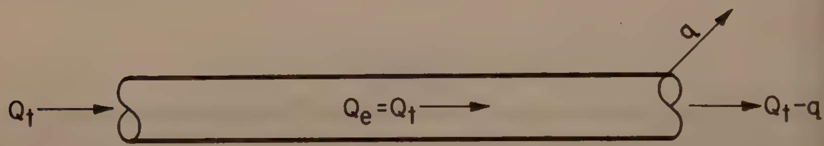

FIG. 1.—CONSERVATIVE SIMPLIFICATION

That this assumption is conservative is illustrated in Table 1, which ind cates that the friction losses so computed for a line may vary from 0% to much as 185% greater than that which actually exists for the condition of se vice connections uniformly spaced along the line from its inlet to its outlet

TABLE 1.—PERCENTAGE ERROR RESULTING FROM ASSUMPTION THAT QUANTITY WITHDRAWN IS REMOVED FROM OUTLET END

Number of Connections	Ratio of Quantity Withdrawn (q) to Total Inlet Flow (Q_t)										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0
2	0.0	4.7	9.7	15.0	20.3	25.9	31.9	37.9	44.1	50.3	56
4	0.0	7.3	15.1	23.9	33.1	43.2	54.5	68.0	78.9	91.7	105
10	0.0	8.8	18.6	31.4	42.1	56.0	71.5	88.2	107.5	127.1	147
20	0.0	9.3	19.9	32.7	45.3	60.5	77.9	97.3	121.5	141.6	165
∞	0.0	10.0	21.5	33.9	48.3	65.3	84.6	106.3	130.7	157.6	185

computed by the summation of losses between each connection by rigorous cor putation in the manner shown in Fig. 3 (to be presented subsequently). In th discussion it has been assumed that (a) the Hazen-Williams flow equation, a plies, wherein the friction loss is assumed to be a function of the 1.85 pow of the discharge; (b) the connections are uniformly spaced along the main; a

c) the withdrawals at each service connection are equal. Turbulence losses t each of the connections have been neglected.

EQUIVALENT DISCHARGE AND LENGTH

While conservation is undoubtedly desirable in water supply engineering, alues so obtained may be unrealistic. One alternate approach might be (a) he determination of an equivalent flow (Q_e) which would give the same friction oss as that which is actually encountered; or (b) the determination of a corection factor (K'), which might be applied to the friction loss computed in the onventional manner or to the length of the line to determine the true loss. In his latter case, the equivalent length of the pipe would be equal to K' L.

INFINITE NUMBER OF SERVICE CONNECTIONS

With an infinite number of connections, the flow at any point x in a line of ength L is equal to Q_t, the total incoming flow, less a proportion, x/L, of the

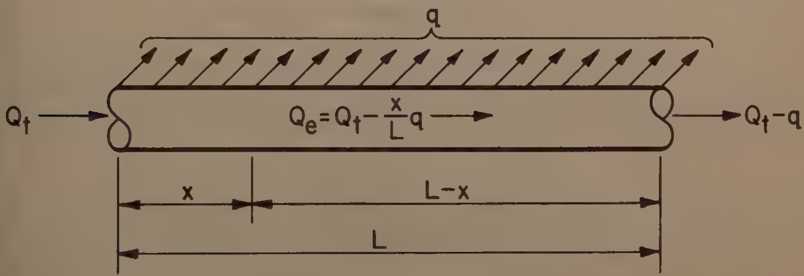

FIG. 2.—INFINITE NUMBER OF CONNECTIONS

uantity withdrawn q—or Q_t - (x/L)q. This is illustrated in Fig. 2. If the fricon loss, h_f, is a function of $Q^{1.85}$, then:

$$h_f = K\,L\,Q_e^{1.85} = \int_0^L K\,dx\left(Q_t - \frac{x}{L}\,q\right)^{1.85} = K\,L\,K'\,Q_t^{1.85} \quad \ldots\ldots (1)$$

'here Q_e is the equivalent discharge in the pipe necessary to develop the acal friction loss and K' is the correction to be applied to the length or to the iction loss computed on the basis of the total incoming flow in order to dermine the true friction loss:

$$K' = \left(\frac{Q_e}{Q_t}\right)^{1.85} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

nd

$$Q_e = (K')^{1/1.85}\,Q_t = (K')^{0.54}\,Q_t \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Expanding by the binomial theorem and substituting in the original equation:

$$h_f = K \int_0^L dx \, (Q_t^{1.85} + 1.85\, Q_t^{0.85} \left(- \frac{x}{L} q\right) + \frac{(1.85)(0.85)}{2} \, Q^{-0.15} \left(- \frac{x}{L} q\right)^2$$

$$+ \frac{(1.85)(0.85)(-0.15)}{(2)(3)} \, Q^{-1.15} \left(- \frac{x}{L} q\right)^3 + \ldots] \ldots\ldots\ldots\ldots (4$$

and

$$h_f = K\, L\, Q_t^{1.85} \, [1 - 0.9250\left(\frac{q}{Q_t}\right) + 0.2621 \left(\frac{q}{Q_t}\right)^2$$

$$+ 0.009820\left(\frac{q}{Q_t}\right)^3 + 0.0002260\left(\frac{q}{Q_t}\right)^4 + 0.0000809 \left(\frac{q}{Q_t}\right)^5 + \ldots] \ \ldots \ (5$$

The limits of the equation may be developed as follows:

$$h_f = K \int_0^L \left(Q_t - \frac{x}{L} q\right)^{1.85} dx = K\, Q_t^{1.85} \int_0^L \left(1 - \frac{x}{L} \frac{q}{Q_t}\right)^{1.85} dx \ \ldots \ (6$$

$$h_f = K\, Q_t^{1.85} \left[-\left(\frac{1}{2.85}\right)\left(\frac{L\, Q_t}{q}\right)\left(1 - \frac{x}{L} \frac{q}{Q_t}\right)^{2.85} \right]_0^L \ldots\ldots (7$$

and

$$h_f = K\, L\, Q_t^{1.85} \left(\frac{1}{2.85\, q/Q_t}\right)\left[1 - \left(1 - \frac{q}{Q_t}\right)^{2.85}\right] \ldots\ldots\ldots (8$$

or

$$K' = \frac{1}{2.85\, q/Q_t} \left(1 - \left(1 - \frac{q}{Q}\right)^{2.85}\right) \ldots\ldots\ldots\ldots (9$$

The values of Q_e and K' computed from Eq. 9 and illustrated in Table 2 appl only to the condition in which there are essentially an infinite number of con nections. Table 2 illustrates the variations of these values as a function

TABLE 2.—VARIATIONS OF Q_e/Q_t and K' FOR INFINITE NUMBER OF CONNECTIONS

q/Q_t	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.(
Q_e/Q_t	1.000	0.951	0.903	0.854	0.807	0.762	0.718	0.676	0.636	0.598	0.5
K'	1.000	0.910	0.826	0.747	0.673	0.605	0.541	0.486	0.434	0.389	0.3

q/Q_t, the ratio of the quantity withdrawn to the quantity entering the section. These show that the values of Q_e/Q_t range from 0.568 to 1.000; and the values of K' range from 0.351 to 1.000.

FINITE NUMBER OF CONNECTIONS

Values of friction loss for finite values of the number of connections n may be computed by expansion of a series assuming takeoffs uniformly spaced L/n

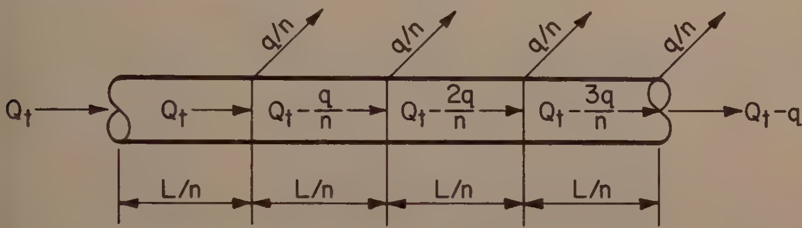

FIG. 3.—FINITE NUMBER OF CONNECTIONS

distance apart and withdrawing a flow q/n at each takeoff. The procedure is illustrated in Fig. 3 and the equations which follow:

If the relationship $k_f = K\,L\,Q_e^{1.85} = K\,L\,Q_r^{1.85}\,K'$ is expanded as in Fig. 3, then

$$h_f = K\left[\frac{L}{n}\left(Q_t^{\,1.85} + \frac{L}{n}\left(Q_t - \frac{q}{n}\right)^{1.85} + \frac{L}{n}\left(Q_t - \frac{2\,Q}{n}\right)^{1.85} + \ldots + \frac{L}{n}\left(Q_t - \frac{(n-1)(q)}{n}\right)^{1.85}\right] \quad \ldots\ldots\ldots\ldots (10)$$

$$h_f = \frac{K\,L}{n}\,Q_t^{1.85}\left[1 + \left(1 - \frac{q}{Q_t\,n}\right)^{1.85} + \left(1 - \frac{2\,q}{Q_t\,n}\right)^{1.85} + \left(1 - \frac{3\,q}{Q_t\,n}\right)^{1.85} + \ldots + \left(1 - \frac{(n-1)\,q}{Q_t\,n}\right)^{1.85}\right. \quad \ldots\ldots\ldots (11)$$

or

$$\frac{K\,L}{n}\,Q_t^{1.85}\sum_{L=0}^{i=n-1}\left[1 - \frac{i\,q}{Q_t\,n}\right. \quad \ldots\ldots\ldots\ldots (12)$$

$$h_f = \frac{K\,L}{n}\,Q_t^{1.85}\left[n + 1.85\left(-\frac{q}{Q_t\,n}\right)\sum(1 + 2 \ldots\ldots + (n-1)) + \frac{(1.85)(0.85)}{2}\left(-\frac{q}{Q_t\,n}\right)^2\sum(1^2 + 2^2 + \ldots + (n-1)^2 + \ldots\right] \ldots (13)$$

and

$$h_f = K\,L\,Q_t^{1.85}\left[1 - \frac{1.85}{n^2}\left(\frac{q}{Q_t}\right)\sum(1 + 2 + \ldots + (n-1)\right.$$

$$\left. + \frac{0.7862}{n^3}\left(\frac{q}{Q_t}\right)^2 \sum(1^2 + 2^2 + \ldots + (n-1)^2) + \ldots\right] \ldots\ldots (14)$$

It should be noted that Eq. 12 reduces to Eq. 9 as n approaches infinity. The values of Q_e/Q_t and K' are found to lie within the same limits of 0.568 to 1.000 and 0.351 to 1.000 respectively. These values are given in Tables 3 and 4 and are plotted in Figs. 5 and 6.

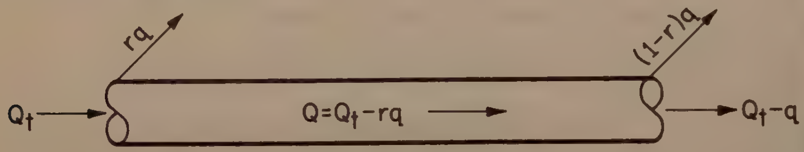

FIG. 4.—BEGINNING AND END WITHDRAWALS

TABLE 3.—VARIATION OF Q_e/Q_t VS. NUMBER OF CONNECTIONS

Number of Connections	Ratio of Quantity Withdrawn (q) to Total Inlet Flow (Q_t)										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	1.000	0.976	0.951	0.928	0.904	0.883	0.861	0.840	0.821	0.802	0.785
4	1.000	0.963	0.927	0.894	0.859	0.824	0.795	0.757	0.730	0.703	0.677
10	1.000	0.955	0.912	0.869	0.827	0.786	0.747	0.711	0.675	0.642	0.613
20	1.000	0.952	0.906	0.861	0.817	0.774	0.733	0.693	0.651	0.621	0.590
∞	1.000	0.951	0.903	0.854	0.807	0.762	0.718	0.676	0.636	0.598	0.568

TABLE 4.—VARIATION OF K' VS. NUMBER OF CONNECTIONS

Number of Connections	Ratio of Quantity Withdrawn (q) to Total Inlet Flow (Q_t)										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	1.000	0.955	0.912	0.870	0.831	0.794	0.758	0.725	0.694	0.665	0.638
4	1.000	0.933	0.870	0.811	0.757	0.698	0.647	0.596	0.559	0.521	0.480
10	1.000	0.919	0.843	0.771	0.704	0.641	0.583	0.531	0.482	0.440	0.402
20	1.000	0.913	0.834	0.758	0.688	0.623	0.562	0.507	0.451	0.414	0.377
∞	1.000	0.910	0.826	0.747	0.673	0.605	0.541	0.486	0.434	0.389	0.351

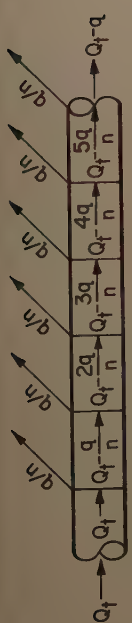

FIG. 3.—ACTUAL CONDITIONS

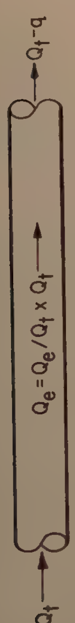

FIG. 5A.—SIMPLIFICATION (NO ERROR)

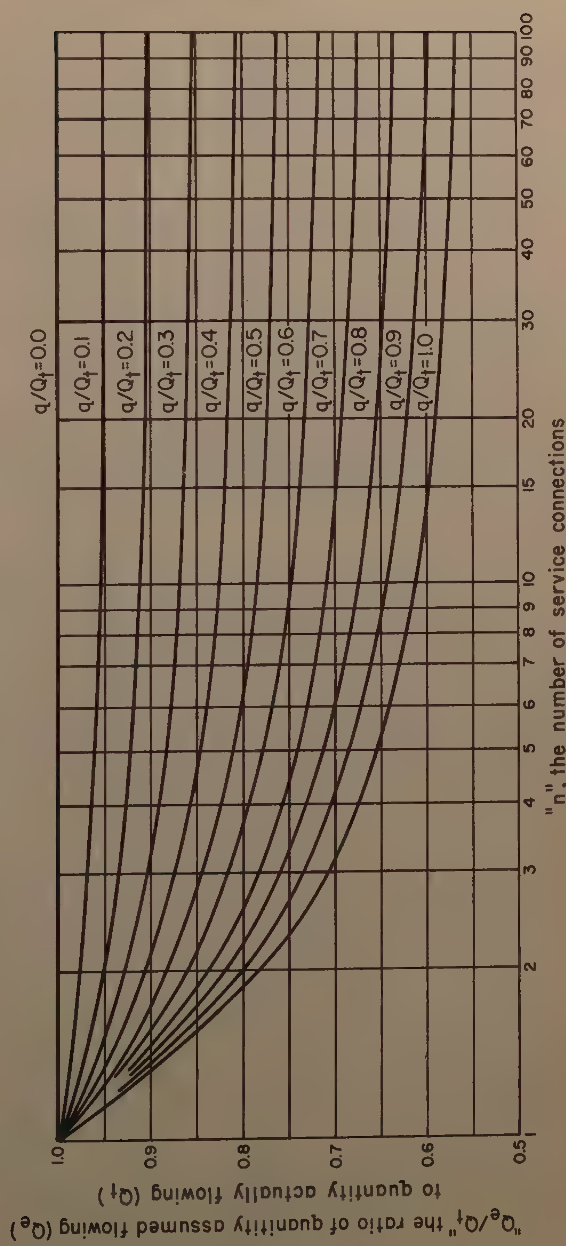

FIG. 5.—NUMBER OF SERVICE CONNECTIONS VS. INLET FLOW CORRECTIONS

APPLICATIONS

If the system is of the dead-end type, it is the usual procedure to determine the flow withdrawn from each section of the main q and, by accumulation, the flow entering each section of the main Q_t. The ratio q/Q_t and the number of services, n, may be used to determine the equivalent length of the line (K' L), the true head loss, or the equivalent discharge through the main (Q_e) for the correct friction loss.

Example.—If 2.0 mgd enter the line and 0.6 mgd are withdrawn along the line from ten connections; the pipe is 1,000 ft long and C = 120: $Q_t = 2.0$, $q = 0.6$; $n = 10$; $q/Q_t = 0.3$; $L = 1{,}000$. From Fig. 5 Q_e/Q_t is determined as 0.869 and $Q_e = 0.869 \times 2.0 = 1.738$ mgd for correct friction computation. Also, h_f = 4.16 ft/1,000 ft or 4.16 ft. The same result may be obtained from Fig. 6 from which K' = 0.771 and L_e = K' L or 0.771 x 1,000 = 771 ft for correct friction computation; h_f = 5.40 ft/1,000 ft x 0.771 or 4.16 ft.

Although this method is useful and precise with regard to dead-end systems, it has very definite drawbacks when applied to loop systems. The drawback, of course, is that in using the Hardy Cross method or a computor, the flows at the beginning and end of each line are not known in advance; and since q/Q_t is not known, the quantity removed at the various junctions cannot be established.

RATIO REMOVED AT THE BEGINNING OF THE LINE

An approximation used in European practice[3,4] and to a limited extent in the United States, is to assume that 45% of the withdrawal takes plate at the beginning of the line and 55% at its end. The various values of K' for this approximation may be computed as follows:

Let r q be the amount of flow withdrawn at the beginning of the section and (l-r) q be the amount withdrawn at the end of the section. The flow in the pipe would, therefore, be equal to Q_t - r q and

$$(h_f)_{approx} = L\,K\,Q_e^{1.85} = L\,K\,(Q_t - r\,q)^{1.85} = L\,K\,Q_t^{1.85}\,(K')_{approx} \quad \text{. . (15)}$$

$$(h_f)_{approx} = L\,K\,(Q_t^{1.85} + 1.85\,Q_t^{0.85}\,(-r\,q) + \frac{(1.85)(0.85)}{2}\,Q_t^{-0.15}\,(-r\,q)^2 + \ldots] \quad \ldots\ldots\ldots\ldots (16)$$

$$(h_f)_{approx} = L\,K\,Q_t^{1.85}\left[1 - 1.85\left(-\frac{r\,q}{Q_t}\right) + \frac{(1.85)(0.85)}{2}\left(-\frac{r\,q}{Q_t}\right)^2 + \frac{(1.85)(0.85)(-0.15)}{(2)(3)}\left(-\frac{r\,q}{Q_t}\right)^3 + \ldots\right] \quad \ldots\ldots\ldots\ldots (17)$$

FIG. 3.—ACTUAL CONDITIONS

FIG. 6A.—SIMPLIFICATION (NO ERROR)

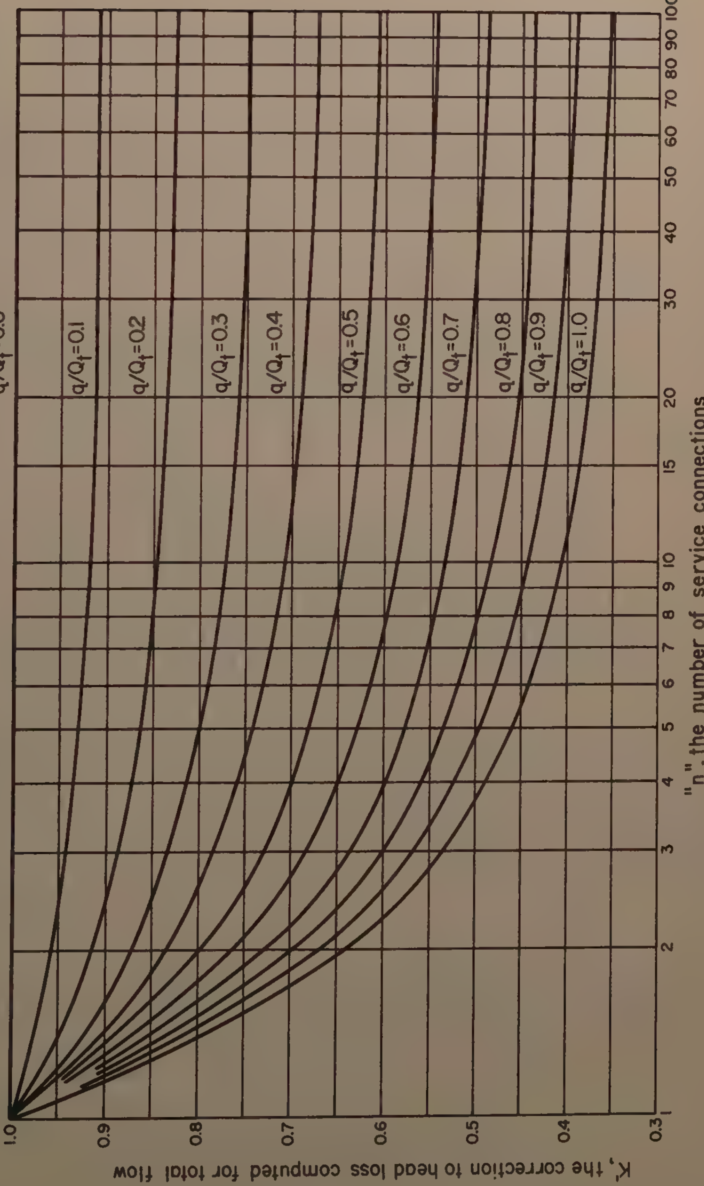

FIG. 6.—NUMBER OF SERVICE CONNECTIONS VS. FRICTION CORRECTION FACTOR

$$(h_f)_{approx} = L\,K\,Q_t^{1.85}\left[1 - 1.850\left(\frac{r\,q}{Q_t}\right) + 0.7862\left(\frac{r\,q}{Q_t}\right)^2 + 0.0393\left(\frac{r\,q}{Q_t}\right)^3 + \ldots\right] \quad (18)$$

Values of (K') approx were computed from Eq. 18 and the values compared with the actual values of K' computed by Eq. 12. The percentage errors were then computed and presented in Table 5. It will be noted that the error ranges from -5.7% to +3.5% for an infinite number of connections to -66.9% to 0.0% for a single connection. Accordingly, this procedure is correct only when there are a large number of connections since the values obtained for smaller numbers of connections are all on the low side.

TABLE 5.—PERCENTAGE ERRORS RESULTING FROM APPROXIMATION THAT 45% OF QUANTITY IS WITHDRAWN FROM BEGINNING OF MAIN AND 55% FROM ITS END

Number of Connections	Ratio of Quantity Withdrawn (q) to Total Inlet Flow (Q_t) = q/Q_t										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	0.0	-8.5	-17.0	-24.5	-31.0	-37.5	-44.0	-50.3	-56.3	-61.5	-66.9
2	0.0	-4.2	- 9.0	-13.2	-17.0	-21.0	-26.1	-31.4	-37.0	-42.1	-48.2
4	0.0	-1.8	- 4.5	- 6.4	- 8.1	-10.5	-13.4	-16.6	-21.9	-26.1	-31.9
10	0.0	-0.4	- 1.5	- 1.8	- 2.0	- 2.5	- 4.0	- 6.4	-11.2	-12.5	-17.9
20	0.0	0.0	- 0.5	+ 0.3	+ 0.3	+ 0.3	- 0.4	- 2.0	- 3.1	- 4.6	-12.2
∞	0.0	+0.5	+ 0.5	+ 1.1	+ 2.5	+ 3.3	+ 3.5	+ 2.3	- 0.8	- 3.0	- 5.7

The conservative simplification in which r is assumed as zero (that is, all of the quantity withdrawn from a section is assumed to be withdrawn at its end) results in positive errors of from 0 to 185%. In contrast, when r is assumed as 0.45 (that is, 45% of the quantity withdrawn from a section is assumed to be withdrawn at the beginning of the section and 55% at its end) results in errors varying from +3.5% to -66.9%.

EXACT VALUES OF r AS A FUNCTION OF q/Q_t AND n

An exact value of r, the ratio assumed withdrawn at the beginning of the section, can be readily computed from the expression $Q_e = Q_t - r\,q$, where Q_e is the discharge which will produce the correct friction loss:

$$r = \frac{Q_t - Q_e}{q} = \frac{1 - \dfrac{Q_e}{Q_t}}{q/Q_t} \quad (19)$$

The ratio Q_e/Q_t is the ratio of the equivalent discharge to the total discharge entering the pipe and the ratio q/Q_t is the ratio of the quantity withdrawn in a

ection to the total flow entering the section. These terms were developed reviously.

From the values of Q_e/Q_t, computed from Eq. 12 and tabulated in Table 3, e corresponding values of r have been tabulated in Table 6 for various values f q/Q_t.

From the data in Table 6 it is readily apparent that r is more a function of than of q/Q_t. The variation of q/Q_t for any value of n varies as a ratio of

TABLE 6.—VALUES OF r AS A FUNCTION OF n AND q/Q_t

Number of onnections	Ratio of Quantity Withdrawn (q) to Total Inlet Flow (Q_t)										
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.000	0.247	0.245	0.242	0.235	0.234	0.232	0.229	0.224	0.220	0.215
4	0.000	0.368	0.364	0.360	0.356	0.352	0.347	0.342	0.336	0.330	0.323
10	0.000	0.450	0.440	0.437	0.432	0.428	0.422	0.413	0.406	0.398	0.387
20	0.000	0.480	0.470	0.463	0.458	0.452	0.445	0.439	0.436	0.421	0.410
∞	0.000	0.495	0.485	0.483	0.482	0.478	0.470	0.463	0.456	0.447	0.432

15 to 1 for the ranges $q/Q_t = 0.1$ to $q/Q_t = 1.0$. Obviously, a universal value r r cannot be used if a high degree of accuracy is required since r varies om 0.215 to 0.495 for the values shown in Table 6.

APPROXIMATE VALUES OF r AS A FUNCTION OF n ALONE

A study of the data illustrating the narrow range in r as a function of q/Q_t ggested the possibility of approximating r as a function of n alone. Accordgly, values of $(K')_{approx}$ were selected such that the negative error in the

TABLE 7.—SELECTION OF TRIAL VALUES OF r

umber of nnections	K' Minimum	0.97K' Minimum	r Trial
1	1.000	--	0.00
2	0.638	0.619	0.23
4	0.486	0.471	0.33
10	0.403	0.391	0.40
20	0.377	0.366	0.42
∞	0.351	0.341	0.44

mputation of friction loss would not exceed 3%. Corresponding values of $/Q_t$ were computed from the expression $K' = \left(\frac{Q_e}{Q_t}\right)^{1.85}$ and trial values of r re computed from Eq. 19 for each of the values of n. Minor adjustments were sequently made to spread the error more uniformly over the positive and gative range of errors. The trial values of r, which would be considered apximately independent of q/Q_t, are shown in Table 7.

Using the trial values of r, the corresponding values of $(K')_{approx}$ were computed from Eq. 18 and these values compared with the true values of K computed from Eq. 12. Errors were then determined for the complete range of values of q/Q_t from 0.0 to 1.00 and the range of n from 1 to infinity. The results are summarized in Table No. 8.

The results of the study indicate that the approximation method using the suggested values of r will yield results ranging from -3.5% to +5.5%, well within the range of accuracy of the pipe friction equations. Fig. 7 graphically illustrates the relationship between the number of connections, n, and the ratio r, for the minimum range of error.

TABLE 8.—SUGGESTED VALUES OF r

Number of Connections	Suggested Values of r	Maximum Range of Error in K	
		(-)	(+)
1	0.00	-0.0%	+0.0%
2	0.22	-1.1%	+0.7%
4	0.33	-1.8%	+3.7%
10	0.40	-3.5%	+3.3%
20	0.42	-2.6%	+4.6%
∞	0.44	-2.0%	+5.5%

A least-squares correlation indicates that the empirical equation of best fit is an exponential

$$r = 0.44 \left(1 - e^{-0.692\,(n-1)^{0.58}}\right) \quad \ldots\ldots\ldots\ldots\ (20)$$

Analysis of the deviations using the Student "t" test indicates a significance of greater than 99.9%. However the equation is too complex for routine use and graphical selections of values of r appear more useful.

APPLICATIONS

Since this procedure is independent of the ratio q/Q_t, it is therefore particularly suitable for use in network analysis, since the ratio need not be known in advance of the final network computations. However, adjustments may be necessary if the wrong direction of flow was assumed for the line. The procedure is simple to apply.

Example.—If 0.6 mgd are withdrawn from ten connections along 1,000 ft of 12 in. pipe, C = 120, n = 10. From Fig. 7, r is determined as 0.40, and the amount assumed to be withdrawn at the beginning of the line is 0.40 x 0.6 mgd or 0.24 mgd, and at the end of the line, (1 - 0.40) x 0.6 mgd or 0.36 mgd. The calculations then proceed in the usual manner. If, after the calculations, the incoming flow is determined as 2.0 mgd, the friction loss in the pipe is determined on the basis of $Q_t - r\,q$ or 2.0 mgd - 0.24 mgd or 1.76 mgd. The friction loss is therefore 4.25 ft, which compares within 2.2% of the correct value of 4.16 ft.

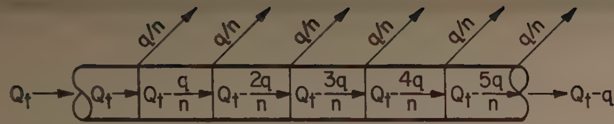

FIG. 3.—ACTUAL CONDITIONS

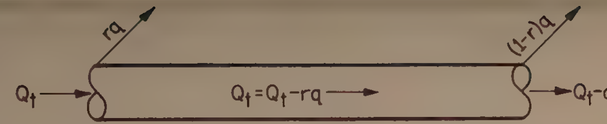

FIG. 4.—SIMPLIFICATION (ERROR 3.5% TO 5.5%)

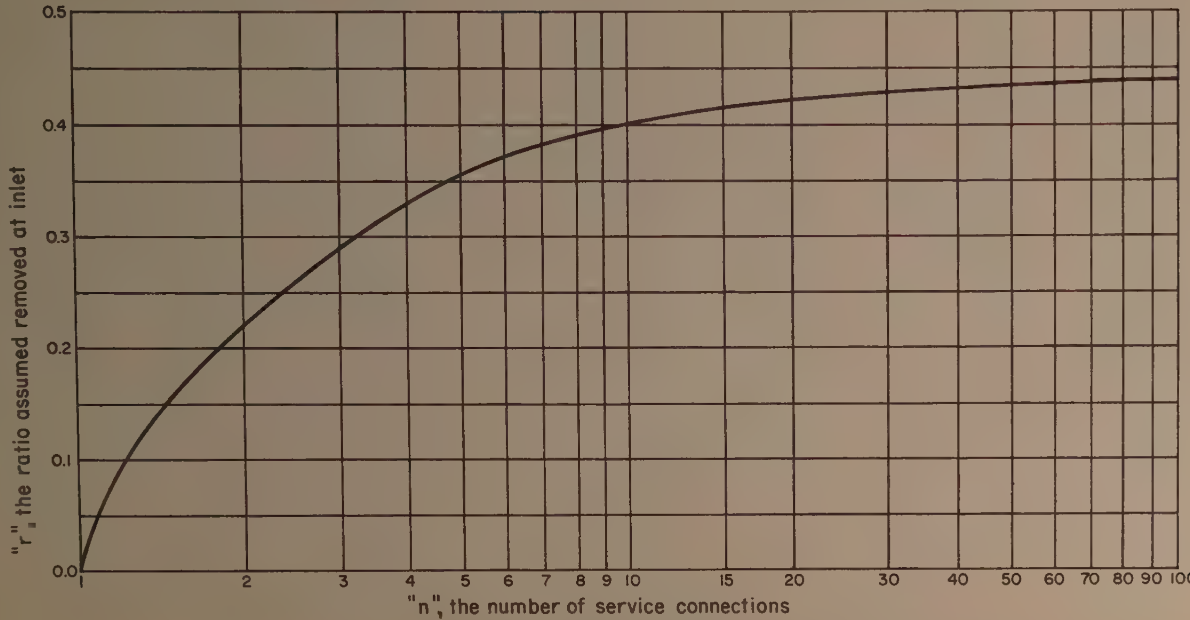

FIG. 7.—NUMBER OF SERVICE CONNECTIONS VS. RATIO ASSUMED REMOVED AT INLET TO A WATER MAIN SECTION

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

RADIOACTIVE TRACERS IN HYDROMETEOROLOGY[a,b]

By L. Machta[1]

SYNOPSIS

Atmospheric motion is studied with the aid of radioactive tracers. Radio-ctive materials are used to improve the meteorologist's ability to track the th of masses of water in the atmosphere. The most promising tracer seems be tritium but the large number of sources of tritium complicates its appli-tion to hydrological problems.

INTRODUCTION

It is most reasonable that the study of the movement of atmospheric water por be conducted with water vapor itself as the tracer. Almost all hydro-eteorological research follows such procedures. But, to the conventional ensing instrument one water vapor molecule is no different from another. racking a specific mass of water in the atmosphere simply by successive oservations of atmospheric humidity is often quite difficult. Rather, the path a mass of water vapor is normally computed from the wind observations. ut today other possibilities are available since water vapor molecules can be entified by special labels; the isotopes of oxygen and hydrogen. In addition, air mass containing the moisture can be identified and tracked by radio-tive aerosols or gases.

Note.—Discussion open until September 1, 1960. To extend the closing date one onth, a written request must be filed with the Executive Secretary, ASCE. This paper part of the copyrighted Journal of the Hydraulics Division, Proceedings of the Amer-an Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

a Research supported by the Div. of Biology and Medicine of the U. S. Atomic En-gy Comm.

b Presented at the October 1959 ASCE Convention in Washington, D. C.

1 U. S. Weather Bur., Washington, D. C.

In this paper, we shall review some of the information dealing with tritium and other radioactive tracers in the atmosphere.

RADIOACTIVE TRACERS FOR AIR MASSES

Water vapor in the atmosphere spreads with the air so long as it remains uncondensed. Horizontal transport and mixing by the atmosphere are vital elements in the understanding of the atmospheric link of the hydrologic cycle.

Air masses have been tagged by fission products from weapon tests. The measurements of this radioactivity, as nuclear clouds move across the United States from the Nevada Test Site, provide both the trajectories and diffusion rates. We have been able to verify trajectories[2] for distances of 1,000 miles or more for the first time at altitudes below 30,000 ft. It turns out that because of better wind observations at lower altitudes our ability to reconstruct the air paths below 30,000 ft is better than might have been guessed from the verification of balloon flights above 30,000 ft.

But more important is the fact that one now has more knowledge of the rate of lateral diffusion for the atmosphere.[2,3] Previously, there were two other sources of information on horizontal mixing for air trajectories to great distances: the first was derived from the spread of moisture itself[4] and the second based on the spread of pairs of constant level balloons.[5] The three sets of observations from moisture, from balloons, and from radioactive clouds make an interesting pattern. The separation of balloon pairs suggests a coefficient of horizontal mixing of about 10^7 cm^2 sec^{-1}; the spread of radioactive clouds, about 10^8 cm^2 sec^{-1}; and the spread of moisture, about 10^9 cm^2 sec^{-1}. An explanation of the reason for the difference in these diffusion rates may help us understand the nature of mixing in the atmosphere.

It is the writer's view that much of nature's horizontal mixing of water vapor or other gases depends on wind shear and vertical diffusion. This phenomenon is schematically illustrated in Fig. 1. Fig. 1(a) is a top view, looking down on a cylinder which, at $t = 0$, is vertical (for the sake of argument). The atmosphere's winds vary in direction and speed with altitude so that at a later time, $t = t_1$, the cylinder is stretched out. The winds in the upper part of the cylinder were from the southwest and near the bottom, lighter and from the west. Looking down at the top at the later time, we find that each horizontal section of the initial cylinder has grown. The initial size is the inner circle. The cylinder would still have this radius if there were no growth. The closely stippled area, also circular in shape, reflects the growth of the cloud due to true diffusion. In horizontally isotropic turbulence, the spread is the same in all directions. This growth might be derived from the coefficient of horizontal mixing of the order of 10^7 cm^2 sec^{-1} as derived from balloons. The outer cross-hatched area shows the growth due to shear and vertical turbulence. As the cylinder tilts, as seen in Fig. 1(b), it is evident that vertical mixing will bring tracer material upward from below and downward from above. This produces a further growth in the horizontal dimensions of the cylinder. The additional enlargement takes place in the line of the wind-shear vector as seen

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[2] Machta, L., Hamilton, H. L., Jr., Hubert, L. F., List, R. J. and Nagler, K. M. 1957 J. Meteor., 14, 165.
[3] Wilkins, E., 1958, Trans. Am. Geophys. Union, 39, 58.
[4] Miller, J. E., 1948, Met. Pap. New York Univ. 1.
[5] Moore, C. B., Smith, J. R. and Gallswyk, A., 1954, J. Meteor., 11, 167.

n Fig. 1(a). Thus, at $t = t_1$, the cylinder has grown to the dimensions of the uter heavy line. From ordinary measurements of concentration one would be nable to discriminate which part of the growth is due to horizontal diffusion lone and which part is due to the effects of shear and vertical diffusion.

The measurements of the spread of radioactive debris suggests that this explanation is correct. We find that the north-south extent of the cloud at say, 0,000 ft is greater when the trajectories just below and just above 20,000 ft

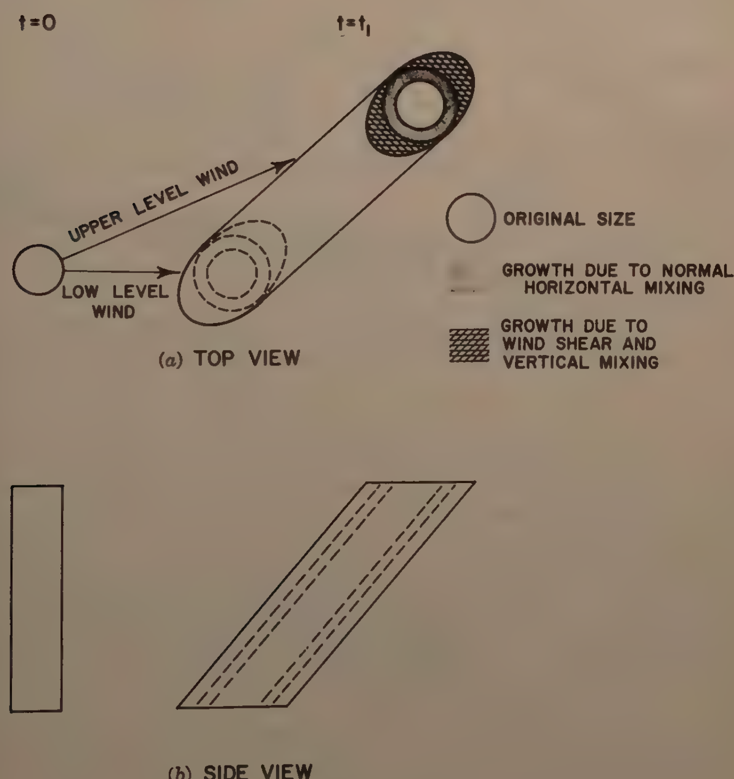

FIG. 1.—VERTICAL COLUMN OF AIR

e to the north and south of the 20,000-ft path as they cross the aircraft sampng line. The coefficient of horizontal diffusion, 10^8 cm^2 sec^{-1}, noted preously, is derived from those cases of very little directional shear. It might noted that these cases were in the minority. Further, all cases have windeed shear; that is, the wind speed changes with altitude. The growth due to ear and vertical diffusion, in the case of speed shear, is along the trajectory ther than laterally and is not detected by the kind of aircraft-sampling operions from which there is information.

The effective diffusion coefficient for moisture appears to be one to two ders of magnitude greater than for balloons or radioactive debris. The coef-

ficient obtained from the lateral spread of radioactive clouds is higher than th spread of pairs of balloons partly because the effect of shear and vertical dif fusion is not entirely eliminated. This indicates that the shear and vertica mixing phenomena are of equal or greater importance than ordinary horizont diffusion. Thus, water vapor clouds will spread horizontally fastest in region where there are the greatest vertical wind shears.

On a larger scale, we have followed particulate debris from the Pacifi atomic tests over almost the entire world. For example,[6] in Fig. 2(b), we ar shown isolines of deposited radioactivity in millicuries per 100 sq miles a ground level during the 30-day period following the March 1, 1954, hydroge

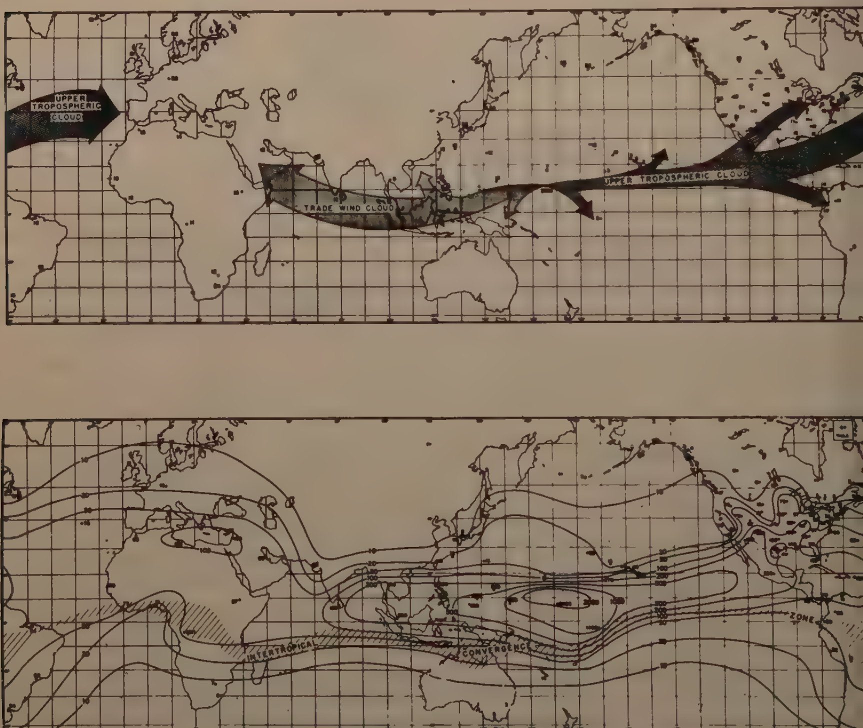

FIG. 2.—HISTORY OF BRAVO CLOUD

bomb test at Bikini. The hatched area of Fig. 2(b) is the intertropical con vergence zone—the meteorological separation between northern and southe hemisphere air. Fig. 2(a) shows the paths of various parts of the nuclear clou for only the first few days. The arrows toward the west show the sense of de bris transport in the trade winds, below about 20,000 ft, while the part of t cloud which moved initially toward North America lay in the 20,000 to 50,00 ft altitude range. All of these paths reflect the east-west or west-east motic which the weatherman believes to be characteristic of the atmosphere's mov ment. The numbers in Fig. 2(a) indicate the number of days between detonati

[6] Machta, L., List, R. J. and Hubert, L. F., 1956, Science, 124, 474.

ınd first ground observation of fission products. The isolines of deposited radioactivity in Fig. 2(b) confirm the more rapid east-west rather than north-south transport of tracer material. Pieces of the cloud are torn from its normal west to east orientation, as in the United States where southerly winds ahead of a low pressure area carried debris into the midwest in about 2 weeks. The slower, general northward spread did not reach the west coast of the United States until weeks later. Only small amounts of radioactivity reached into the southern hemisphere. But equally small amounts are reported in polar latitudes. The clouds from the Hardtack Pacific test series in 1958, the only summer Pacific tests, apparently were carried into the southern hemisphere n greater amounts than spring or fall test clouds.

The tracking of radioactive debris has, by and large, confirmed our notions ıbout the broad movements of air in the troposphere, and some of the details f transport in both time and space.

RADON—A NATURAL RADIONUCLIDE

Radon is a naturally occurring radioactive gas with a 3.7-day half-life which emanates from the rocks and soil of the land. Thus, its source, broadly speaking, is the complement of water vapor in the atmosphere. Where one finds nore water vapor one should also find less radon and vice versa. Insofar as ceanic areas have high moisture and low radon while land areas have the reverse, this is true. But the atmosphere has a limited capacity for moisture, whereas this is not true of radon.

The similarity between radon and water vapor, both evolving from the earth's urface, permits one to deduce information about the vertical flux of water vapor rom observations of radon. Fig. 3 is a plot of radon concentration[7] as the rdinate in Fig. 3(a) and thermal stability in the lower atmosphere as the ordiate in Fig. 3(b). The more positive the number in Fig. 3(b), the more stable he atmosphere. The abscissa is local time of the day. The observations were nade at Harwell, England, and are averaged over a period of several months. The zero gradient line in Fig. 3(b) is the adiabatic lapse rate, negative values re superadiabatic lapse rates, and positive values, more stable than the adiaatic lapse rate. The 0.24° C/76 ft gradient value is the isothermal lapse rate f actual temperature and values greater than 0.24 correspond to temperature nversion conditions.

One notes a diurnal cycle in both curves. Throughout the night, the rate of mission exceeds the rate of upward mixing since, as seen in Fig. 3(b), the air s stable and resists vertical turbulence. With the beginning of solar heating, ne air becomes unstable and the upward flux of radon exceeds the rate of mission from the ground. The radon concentration decreases. The assumption s made that the rate of emanation from the ground is approximately constant nd that the concentration reflects the change in vertical diffusion. This cycle f radon concentration at ground level is no surprise since the diurnal variation of vertical mixing is well known.

Harry Moses, M. ASCE, and his colleagues at Argonne National Laboratory[8] ear Chicago, Ill., have made measurements at the ground (within 1/8 in. of ıe ground) and up to 131 ft on a meteorological tower. The observed diurnal ycle for a summer day with light winds and clèar skies is given in Fig. 4. The

[7] Gale, H. J., Peaple, H. J., 1958, Intern. J. Air Pollution, 1, 103.

[8] Moses, H., Stehney, A. F. and Lucas, H. F., Jr., 1959, Radiological Physics Division Rep., ANL 5967, Argonne National Laboratory, 165.

uppermost curve refers to the 1/8-in. level and the lowermost curve the trace at 131 ft. Something akin to the normal diurnal cycle is observed up to 18.75 ft but not at the 131-ft level. The amplitude of the diurnal cycle diminishes with height and the peak concentration is delayed in time during the night. The wind speeds during the night were very light and there was extreme thermal stability. The almost complete lack of atmospheric turbulence prevents the radon from being transported up to the 131-ft level until 5:00 A.M. At about 5:00 A.M.,

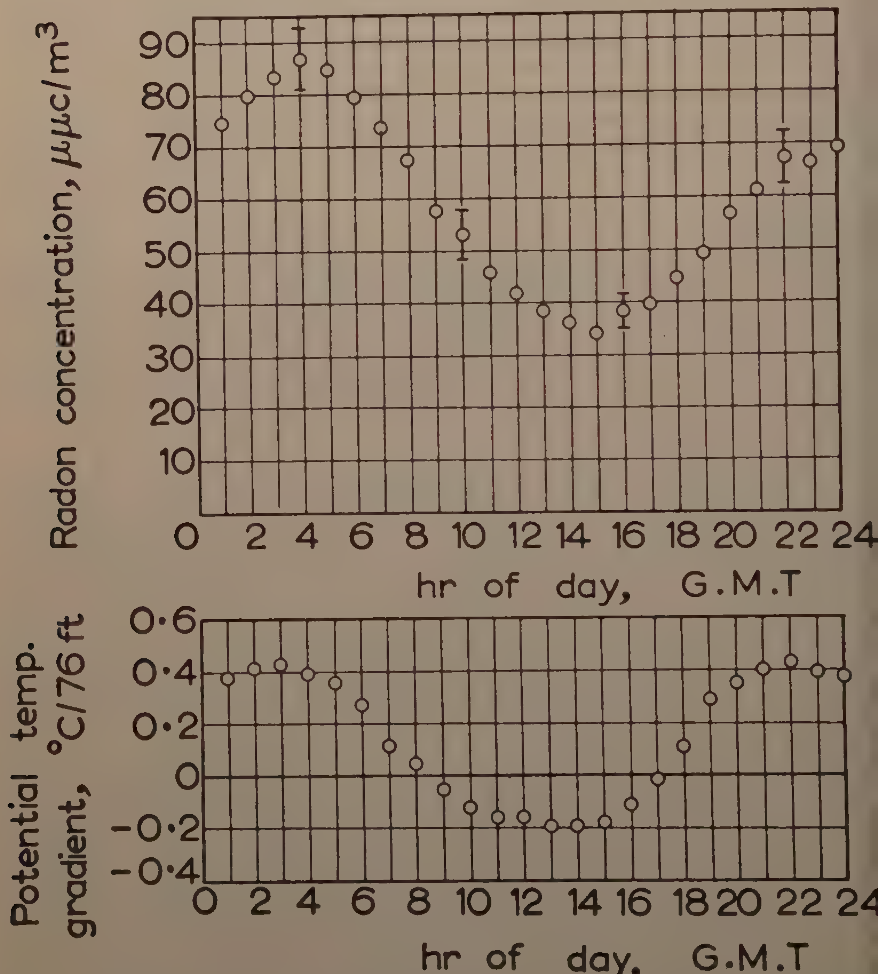

FIG. 3.—DIURNAL CYCLE OF RADON CONCENTRATION AND THERMAL STABILITY

the sun heats the ground and abruptly increases vertical mixing so that, at 131 ft the radon suddenly rises. Then, with deeper layers of the atmosphere involved in the mixing, the upward flux at 131 ft due to turbulence exceeds the growth due to ground emission and the concentrations at all observed levels on the tower diminish after 7:00 A.M. If we had observations at greater altitudes, w

could obtain the rate of growth of the ground layer "blanket" of air during the daytime also. It is clearly greater than 131 ft, of course.

The vertical profile of radon or water vapor emitted from the ground or water can be converted to a measure of vertical turbulence, say, the coefficient of vertical diffusion, if the flux is known. It is here that radon may have an advantage over water vapor, for the radon emission rate is largely independent of atmospheric conditions. The emanation rate can be estimated from these observations and has been given in the literature. The diffusion coefficient derived from the lowermost points at 1/8 in. and 3.17 ft in the middle of the night turns out to be about one order of magnitude larger than molecular diffusion, 1 cm^2 sec^{-1}, one of the lowest values of eddy diffusion in the literature. In the afternoon, the coefficient in this same layer is computed to be about

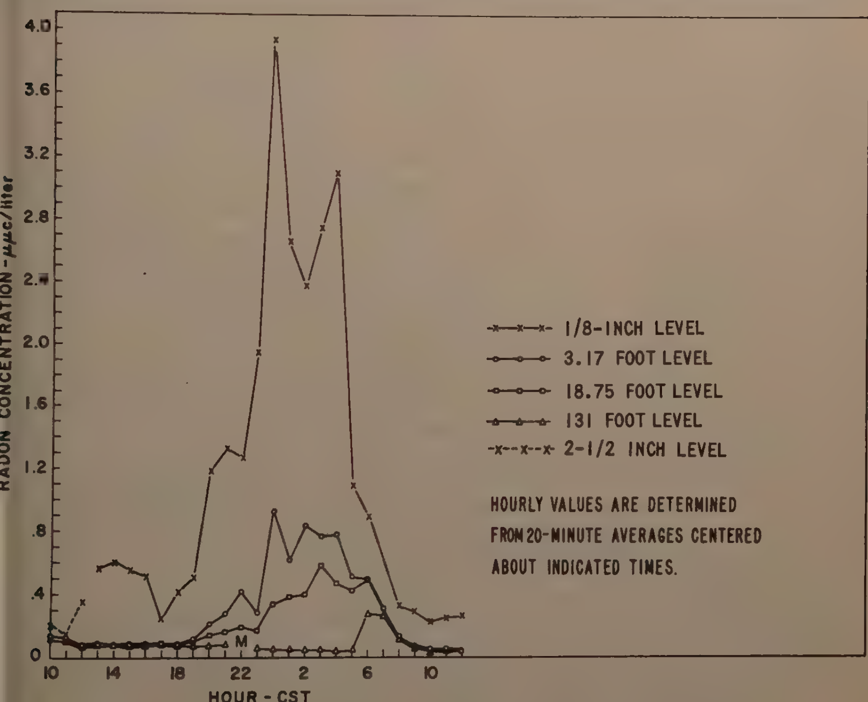

FIG. 4.—RADON CONCENTRATION FOR JULY 16-17, 1958.

00 cm^2 sec^{-1}. The night values in the 3.17 ft to 131 ft layer are also about 00 cm^2 sec^{-1} while daytime values are 1,000 (or more) cm^2 sec^{-1}.

In Fig. 5, similar results for the summer season are given for a cloudy eriod with less incoming or outgoing radiation. Wind speeds were somewhat igher than in the previous experiment and the night time thermal stability was uch less marked. For example, in the previous July case the temperature at 5 ft on the tower was as much as 2° Celsius warmer than at 5.5 ft, but in the resent case, the temperature was never more than 0.5° Celsius warmer. The /8-in. concentration in the afternoon is about the same as in the last case but : night the maximum is less than half that of the July night with its strong

thermal inversion. The coefficient of vertical diffusion during this night is about five times greater than the previous case. The vertical mixing in the 3.17 ft to 131 ft layer was also greater than before and this is reflected in the small diurnal amplitude in the radon concentration at all levels. It is interesting to note that the 1/8-in. concentration decreases fairly steadily from before midnight while the upper levels show an increase during the same interval. These data confirm the differences in measurements of atmospheric properties right above the interface from which the emission occurs as compared with conventional anemometer-height observations.

One must, of course, be cautious in extrapolating any conclusions found from radon measurements to those of evaporating water surfaces: the ground has a different roughness than a water surface, the ground is a better radiator and

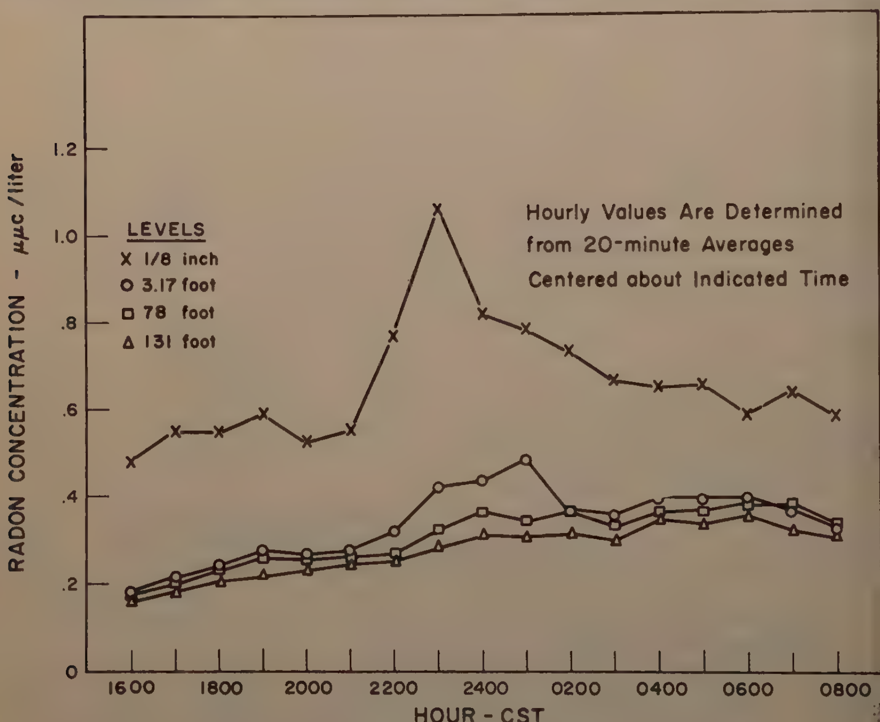

FIG. 5.—RADON CONCENTRATION FOR AUGUST 27-28, 1958.

absorber of heat than the water and has a larger diurnal cycle of stability, and the radon emanation rate may be too variable to attribute all the changes in concentration to atmospheric mixing. The emission from soil will not vary greatly if not too windy days without snow or water cover only are studied. Despite these and other cautions, it is likely that micrometeorology has much to learn from a study of the histories of radon concentration profiles.

NATURAL TRITIUM

Tritium is a radioactive isotope of hydrogen with a half life of 12.5 yr formed by the action of cosmic rays. It is produced in greatest quantities at about

50,000 ft in the atmosphere. Thus, the main source of tritiated water vapor is the upper troposphere or stratosphere rather than the surface, as is the case for ordinary water.

Tritium is normally expressed as T units, that is the number of tritium atoms per 10^{18} hydrogen atoms in the sample. The limit of detectability of tritium is 1 T unit or slightly less. Very recently-disclosed observations of tritium in the stratosphere[9] give tritium concentration as about 10^5 T units, give or take an order of magnitude. Early 1953 rains at Chicago contained about 1 to 10 T units.[10] Oceanic rains at the same time had about 0.5 to 3 T units.[10]

The high stratospheric concentrations reflect the greater production of tritium in the stratosphere. But an additional reason for high concentrations is the very small amounts of moisture in the stratosphere. The stratospheric relative humidity is between about 1% and 15% and the temperatures between -50° and -80° Celsius. If the moisture content were higher, the number of tritium atoms would not change, of course, but the number of hydrogen atoms would increase and the tritium-to-hydrogen ratio would decrease. When the high tritium concentrations are carried into the troposphere, they are diluted with the large amounts of tropospheric moisture and quickly reduce their concentrations to tens or hundreds rather than thousands of T units as in the stratosphere.

Fig. 6 shows the time history of the tritium content of rains at three North American stations: (a) Chicago, (b) New York, N. Y., and (c) Ottawa, Canada.[11,12,13] The abscissa in Fig. 6 is time, increasing to the right, and the ordinate, on a logarithmic scale, is the precipitation tritium concentration in T units. Each collection point is shown with a separate symbol. Although the Canadian laboratory has intercalibrated with the Chicago laboratory to its own satisfaction, these and other data suggest that there is still a systematic difference between the United States and Canadian laboratories with the Canadians greater by a factor of about 2. Fig. 6 shows that the rains after March, 1954 are contaminated by man-made tritium.

It is of some interest to explain the observations of natural cosmic-ray tritium found in rains. For simplicity, consider two reservoirs of water vapor: first, the moisture in the upper troposphere with many tens or hundreds of T units of tritium concentration. This moisture originated from the oceans so long ago that it now can be called "non-recent oceanic tritium." The second is water in the lower atmosphere which has recently evaporated from the oceans. From 1953 measurements of oceanic concentrations of tritium, we know that these water bodies contain 1 T unit or less. All water vapor evaporating from the oceans possesses about the same tritium concentration. Rains at coastal stations or island locations are made up of water which has almost entirely evaporated from the oceans recently and reflects tritium concentrations not much greater than oceanic water. Thus, Hawaiian rains had an average tritium concentration of less than 1 T unit in 1953. But continental rains are mixtures of the two sources of moisture. The bulk of the water is from the relatively recent oceanic source but the greater part of the tritium comes from the upper troposphere moisture reservoir. The day-to-day differences in tritium con-

[9] Hagemann, F., Gray, J., Machta, L. and Turkevich, A., 1959, Science, 130, 542.
[10] Von Buttlar, H. and Libby, W. F., 1955, J. Inorg. Nucl. Chem., 1, 75.
[11] Brown, R. M. and Grummitt, W. E., 1956, Can. J. Chem., 34, 220.
[12] Giletti, B. J., Bazan, F. and Kulp, J. L., 1958, Trans. Am. Geophys. Union, 39, 807.
[13] Begemann, F. and Libby, W. F., 1957, Geochem. et. Cosmich. Acta., 12, 277.

centrations in rains over continents can be attributed in large part to differences in the fractions of water from the two sources. This explanation, first offered by W. F. Libby, accounts for the greater amount of tritium in continental than in oceanic rains even before there were bomb sources of tritium of an appreciable magnitude.

The tritium concentration of oceanic water vapor is known. If one can estimate the tritium concentration of the non-recent oceanic reservoir, it becomes possible to compute the fraction of moisture derived from each source. As a first and rather naive approximation, one can assume the tritium-to-hydrogen ratio in the middle and upper troposphere, the non-recent oceanic source, to be inversely proportional to the amount of moisture at this level relative to the

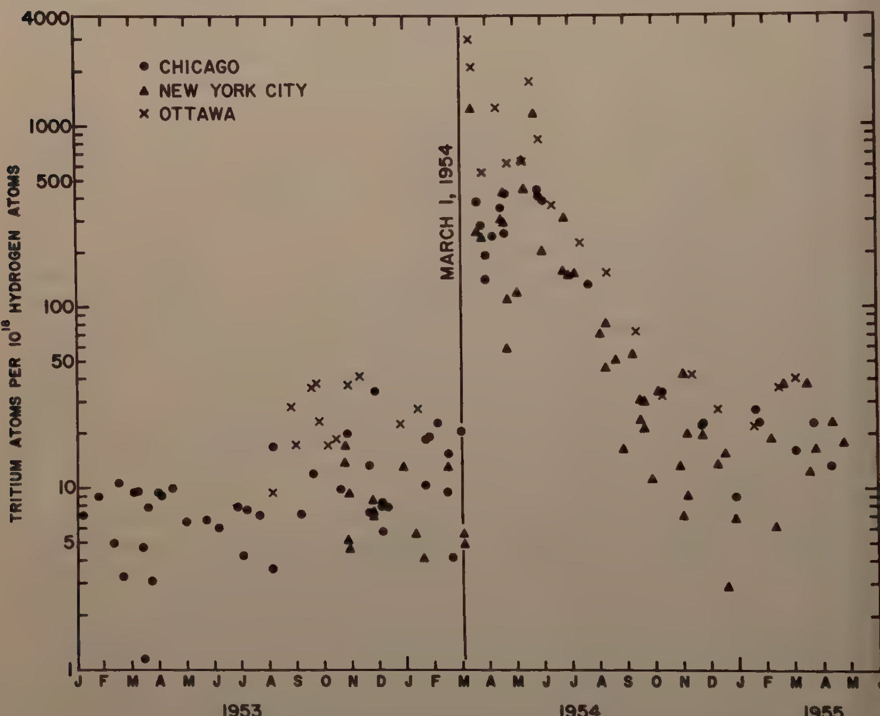

FIG. 6.—TRITIUM CONTENT OF NORTH AMERICAN RAINS

stratosphere or about 100 T units. Simple arithmetic gives the following results: for a rain with 5 T units, about 5% of the water is from this non-recent oceanic source, for 10 T units about 10%, and for 40 T units, the non-recent oceanic source constitutes over 60% of the liquid water. Inspection of Fig. 6 before March 1, 1954, indicates the frequency of occurrence of the various fractions of oceanic and non-recent oceanic moisture. The first half of 1953 may be a more reliable period than the second half of 1953 because of the lesser likelihood of artificial tritium. During this period, 5% to 10% of the rainwater came from non-recent oceanic water. Among the possible shortcomings of this calculation is the assumption that the tritium concentration of a falling raindrop is constant as it passes through water vapor with a different tritium

oncentration. B. Bolin has recently calculated the exchange rate and found it be surprisingly fast.[14]

Some day, many tens of years from now, one may once again be able to use atural tritium when the bomb tritium decays away and becomes thoroughly luted in the oceans. Today, one must deal with tritium from man-made sourcs as well. Actually, for many geophysical purposes, the injection of large mounts of tritium into the atmosphere permits studies which would otherwise ot be possible. Some of these will be discussed herewith.

THE USE OF BOMB-TRITIUM IN HYDROMETEOROLOGICAL PROBLEMS

The data in Fig. 6 show a systematic decrease in concentration after the pring of 1954 which can be approximated by a straight line with a slope correponding to a half-time of about 40 days. That is, the tritium concentration deeases to half its value each 40 days. Since the radioactive decay of tritium 12.5 yr, the decrease is not due to decay. Rather, we are certain it reflects e loss of tritium-rich water vapor from the atmosphere through precipitation. ne water vapor which evaporates from the oceans to replenish the water conins less than 1 T unit, or a negligible concentration of tritium. The transfer the bomb tritium is thus one-way, from atmosphere to ground or oceans via ecipitation.

The 40-day half residence time deduced from the graph must be treated with ution. It does not agree with much shorter estimates made by more conntional methods such as the "7" day half residence time given by H. Lettau.[15] explanation of this difference is necessary. First of all, the tritium conntration should be growing in North America at this time as the tritium difses northward from the Marshall Islands. If this diffusive growth of tritium ncentration is subtracted from the decrease in Fig. 6, the half residence time comes less than 40 days. This fact cannot explain the discrepancy.

There is a difference in the residence time for atmospheric aerosols dending on their altitude or origin. Thus, dust which originates at ground level, d which is tagged, for example, with radon daughter products which are dioactive, suggest a half-residence time of about 1 day[16] while C. Junge and Gustafson[17] estimate about a 3-day half-residence due to the rainout of sea lt particles. On the other hand, the dust which is injected into the upper oposphere (say 20,000 ft to 40,000 ft) by nuclear explosions has been observed be removed with a half time of from about 15 to 30 days.[18] P.S. Goel et al.[19] ind a 30-day half residence in the tropics for dust in the upper troposphere gged with cosmis ray radionuclides.

Removal by precipitation is probably important for both low and high altie dust. But precipitation scavenging is usually limited to levels below 000 ft to 15,000 ft, on the average. Dust or water vapor originating from ground can be removed more quickly than dust or water vapor originating ll above the rain bearing layers which must mix downward before being

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14 Bolin, B., 1958, Proceed. Sec. Intern. Conf. on the Peaceful Uses of Atomic Energy, neva, 18, 336.

15 Lettau, H., 1954, Arch. Meteor. Geophys. Biokl., A, 7, 133.

16 Lehmann, L. and Sittkus, A., 1959, Naturewissenschaft, 1, 9.

17 Junge, C. and Gustafson, C., 1957, Tellus, 9, 164.

18 Stewart, N. G., Osmond, R. G. D., Crooks, R. N., Fisher, E. M. R., 1957, A.E.R.E. /R 2354, Harwell, England.

19 Goel, P. S., Narasappaya, N., Probhakara, C., Thor, R. and Zutshi, P. K., 1959, lus, 11, 91.

washed out of the air. This probably accounts for the discrepancy between the longer residence time derived from the bomb tritium and other estimates. This tritiated moisture was added mainly to the upper troposphere while ordinary water vapor is added from the earth's surface. The same longer tropospheric residence time would apply to natural tritium feeding down from the stratosphere.

The initial impulse of tritium in the rains at New York and Ottawa exceeded 1,000 T units. This means that water vapor from the Marshall Island area contributed measurable moisture to rains in eastern North America. The same kind of arithmetic used to obtain the fraction of non-recent oceanic rain can be applied to the post-March 1954 period. To make such a computation we need the estimate of the tritium concentration of the air contaminated by the bomb tritium. The estimate of the amount of tritium made and available after the atomic cloud stabilizes in the troposphere and the degree of dilution by the time it reaches eastern North America, is the main uncertainty in the calculation.

The fraction of the rain water containing tritium from the Marshall Islands is roughly 5% using such estimates as the writer could make for the bomb-tritium concentration. This number is much higher than would have been expected. Is it possible that moisture from this limited equatorial region contributes so much water to the rain in eastern North America?

The arrival of bomb tritium in eastern North America is also faster than might have been expected but here there is less uncertainty. The first rain with anomolous tritium occurred 10 days after the explosion although if it rained a day or two earlier, we might have had evidence of even earlier arrival. Fast though this is, we have evidence from the rainout of fission products during the Greenhouse test series of Marshall Island air passing over the St. Lawrence River region in 5 days.

CONCLUSIONS

This paper has reported on only a few examples of the use of tracers in the study of atmospheric motions but it is felt that they illustrate the usefulness of this new tool in hydrometeorology. Water can be tracked by both stable and radioisotopes of oxygen and hydrogen and, in fact, a considerable effort has been and is being devoted to this activity. Of these tracers, tritium seems to be one of the more exciting. But what are the prospects for current and future research using tritium?

The atmosphere's tritium is not in equilibrium with the oceans as was the case before man's tritium came on the scene. The large number of tritium sources complicates its application to hydrological problems. As long as the atmospheric high yield nuclear tests are suspended, we can have no new surge of tritium for study. It thus appears that we have either of two prospects for the future; first, make an intensive study of the distribution of tritium in nature so as to be able to constructively use the variability, or, second, add new tritium for small-scale experiments. The writer emphasizes small scale because to "see" new injections of tritium in the atmosphere over large areas of the earth requires the addition of more tritium than we probably are willing to use. At this time (1960), the addition of tracer tritium seems to hold the greatest promise.

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SEDIMENT PROBLEMS OF THE LOWER COLORADO RIVER

By Whitney M. Borland,[1] M. ASCE and Carl R. Miller,[2] M. ASCE

SYNOPSIS

The control of a river to serve the purposes of man alters the river regime thereby creates a number of varied problems. The closure of Hoover Dam he Lower Colorado River, and the subsequent construction of other major ictures downstream, instigated a series of river adjustments that have re-ed corrective or protective measures. Some of the major problems that e been encountered are pointed out, methods of rectification and design siderations are presented, and some of the results that have been obtained given. Some probable future river problems are also briefly examined.

INTRODUCTION

here are many interests to be considered in any river control work in the er Colorado River Basin. State and Federal fish and wildlife agencies, idary commissions, recreation advocates, adjoining property owners, and rs have a vital interest in the river and its use. The material in this paper be restricted to the technical considerations involved in developing river rol operations by the Bureau of Reclamation (USBR), and no attempt will nade to discuss the considerations of the other interested agencies. In the

ote.—Discussion open until September 1, 1960. To extend the closing date one h, a written request must be filed with the Executive Secretary, ASCE. This paper rt of the copyrighted Journal of the Hydraulics Division, Proceedings of the Amer-Society of Civil Engineers, Vol. 86, No. HY 4, April, 1960.

Head, Sedimentation Sect., Hydrology Branch, Div. of Proj. Investigations, Bur. of amation, Denver, Colo.

Supervisory Hydr. Engr., Watershed Tech. Research Branch, Soil and Water Con-ation Research Div., Agric. Research Service, Beltsville, Md. (Formerly with the).

planning of river control work, however, all factors and interests involved ar duly considered, and the plan, when developed, is one that includes the variou river uses and the best engineering plan for present and future river operation

Engineers associated with the planning of the Boulder Canyon Project, o which Hoover Dam is a major feature, realized that the closure of Hoover Dar would cause extensive changes of river regime in the more than 300 miles o river channel, downstream, which extends to the Gulf of California. However the details of these changes and the introduction of new problems could not hav been fully anticipated. Fig. 1 shows the river course and structural contro between Hoover Dam and the Gulf.

Estimates based on available information and data collected prior to th closure of Hoover Dam indicated that the average sediment load being trans ported past the damsite was in excess of 160,000,000 tons or 103,000 acre-f annually. A resurvey of Lake Mead in 1948, and sediment measurements afte 1948, at the Grand Canyon gaging station, have confirmed this estimate. Trap ping this amount of material in Lake Mead and releasing reregulated clea water to the channel downstream was certain to mark the initiation of river channel adjustments. These adjustments, increased by the later completion o the Davis (1950), Parker (1938), and Imperial (1943) Dams, have extended fro Hoover Dam to the Gulf of California, as predicted (Fig. 1). Public Law No 469, enacted by Congress in 1946, established the framework under which th study, planning, and control of the Lower Colorado could be accomplished.

History.—Prior to construction of Hoover Dam, the river moved uncontrolle through deep canyons and wide, alluvial valleys. Rampaging floods occurre in the springtime as the mountain snowpack melted, but the river became relative trickle in the late fall months. The sands and silts in the canyon be and the alluvial fill in the flood plains were alternately shifted, washed ou and replenished as the millions of tons of material moved down to the Gulf. is significant to note that the bulk of the water and sediment carried by th Colorado River is derived from the drainage area above Hoover Dam. Belo Hoover, the river flows through desert terrain where the only water and sedi ment reaching the river is derived from erratic, high intensity, summer storm In the valley flood plains, the river meandered and changed course frequentl destroying a parcel of land in one place and creating new land in another. Dur ing low run-off years, the valley areas suffered from lack of water, and durin high run-off years, the suffering resulted from floods. Historical accounts the fickle nature of the untamed Colorado River are numerous.[3] Fig. 4 illus trates the control of the Lower Colorado River effected by Hoover Dam.

Lake Mead Sedimentation.—As mentioned previously, the sediment load the Colorado River, as it enters Lake Mead, averages about 103,000 acre- annually. This extremely large sediment inflow has led many to believe th Lake Mead would lose its usefulness in a relatively short period of time. Th is certainly far from the truth. Although the sediment inflow is large, t reservoir capacity is also large and, therefore, the annual per cent capaci loss is reasonable. At the present rate of sediment inflow, the 31,250,000 acr ft original capacity of Lake Mead would be reduced to a negligible amount about 300 yr, not considering additional life which may be added due to com paction of deposits and deposition above spillway crest elevation. Howeve with the closure of Glen Canyon Dam, now under construction upstream (Fig. studies indicate the future sediment inflow to Lake Mead will be only one-fou

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3 "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier a Harry F. Blaney, Tech. Bulletin 67, U. S. Dept. of Agric., February, 1928.

FIG. 1.—LOWER COLORADO RIVER, LEES FERRY TO GULF OF CALIFORNIA

of the present rate, and the life of Lake Mead will be extended beyond 1,000 yr. The life of Lake Mead can be expected to be prolonged even further with construction of additional dams and reservoirs between Glen Canyon Dam and Lake Mead, on the main stem of the Colorado River, and on the sediment-carrying tributaries such as the Little Colorado and Paria Rivers (Fig. 1).

AVAILABLE DATA

Data available, in the Lower Colorado River basin, that can be utilized in the determination of existing conditions, and in planning of river rectification work include:

TABLE 1.—SUMMARY OF RESULTS OF CROSS-SECTIONING LOWER COLORADO RIVER[a]

River Reach	Period	River Miles	Quantities (1000 cu yds)[b]
Hoover Dam to Lake Havasu	1935 to 1951	0 - 12.2[c]	- 10,402
		12.2- 25.6	- 13,671
		25.6- 42.7	- 25,716
		42.7- 64.2	- 48,125
		64.2- 82.9	- 39,515
		82.9- 91.8	- 14,401
		91.8- 98.0	+ 8,281
		98.0-111.8	+ 94,737
		111.8-119.4	+ 4,304
Davis Dam to Lake Havasu	1951 to 1956	0 - 5.7[d]	- 2,597
		5.7- 10.9	- 3,541
		10.9- 25.3	- 16,613
		25.3- 30.6	- 1,875
		30.6- 42.7	+ 2,877
		42.7- 53.9	+ 4,495
Parker Dam to Imperial Dam	1937 to 1956	0 - 86[e]	- 190,100[f]
		86 -147	+ 148,970

[a] Report of River Control and Investigations, Lower Colorado River, November 1957. [b] Aggradation = +; degradation = -. [c] River miles below Hoover Dam. [d] River miles below Davis Dam. [e] River miles below Parker Dam. [f] Determined from river cross sections and total sediment transport computations.

a. River cross sections.—These cross sections, extending from Hoover Dam to the International Boundary, are spaced approximately 1.2 miles apart and have been resurveyed at regular intervals since Hoover Dam closure. Results from the resurveys are summarized in Table 1. Cross sections are now being surveyed by the use of echo-sounding equipment where feasible.

b. Suspended sediment sampling.—Suspended sampling was initiated at Yuma in 1911, and at Imperial Dam and Red Cloud (near Taylor's Ferry) in 1933 (Fig. 3). A detailed, periodic, total load sampling program was initiated in 1955, between Davis and Imperial Dams to aid in the design of river channelization work. The total load sampling stations are shown on Figs. 2 and 3

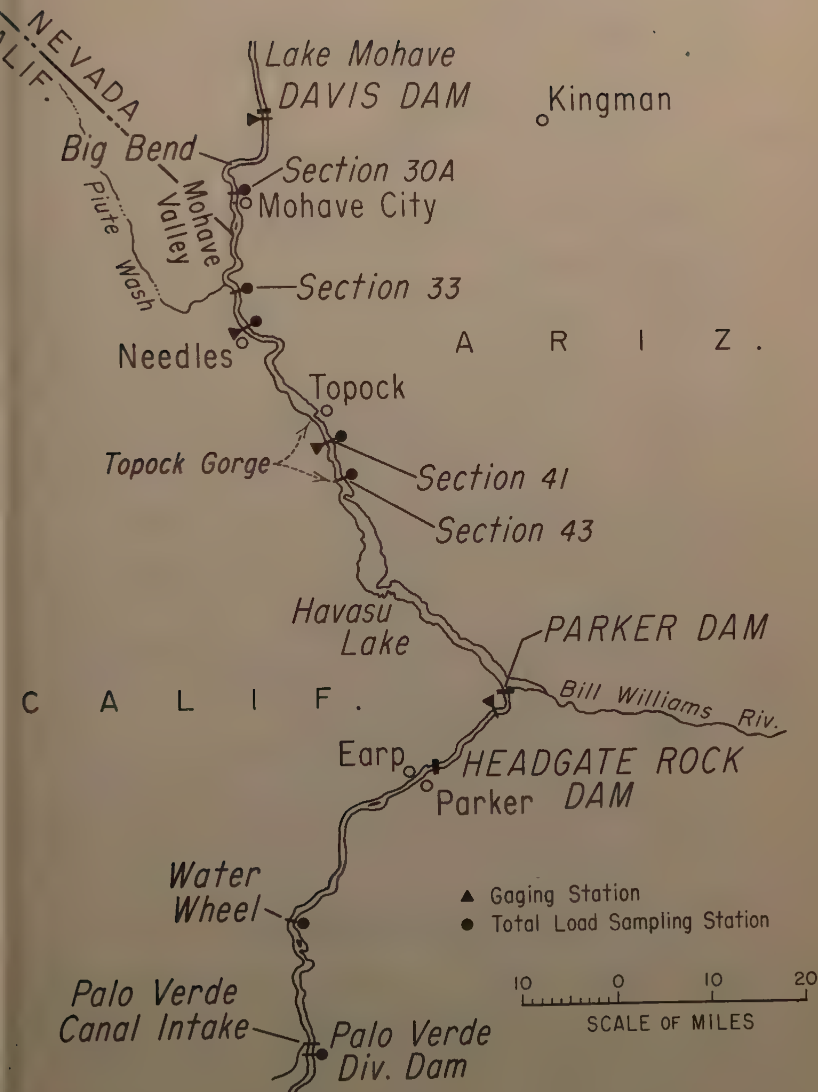

FIG. 2.—LOWER COLORADO RIVER, DAVIS DAM TO PALO VERDE DAM

Total sediment load is presently being determined by the Modified Einstein Procedure.[4] Sampling under the current program is being accomplished by the equal-transit-rate method (ETR). This type of sampling gives a better evaluation of the sediment movement taking place. A sample is collected at each of 25 verticals, spaced across the section. The 25 samples are combined or composited into one sample for analysis. The results of these samples and

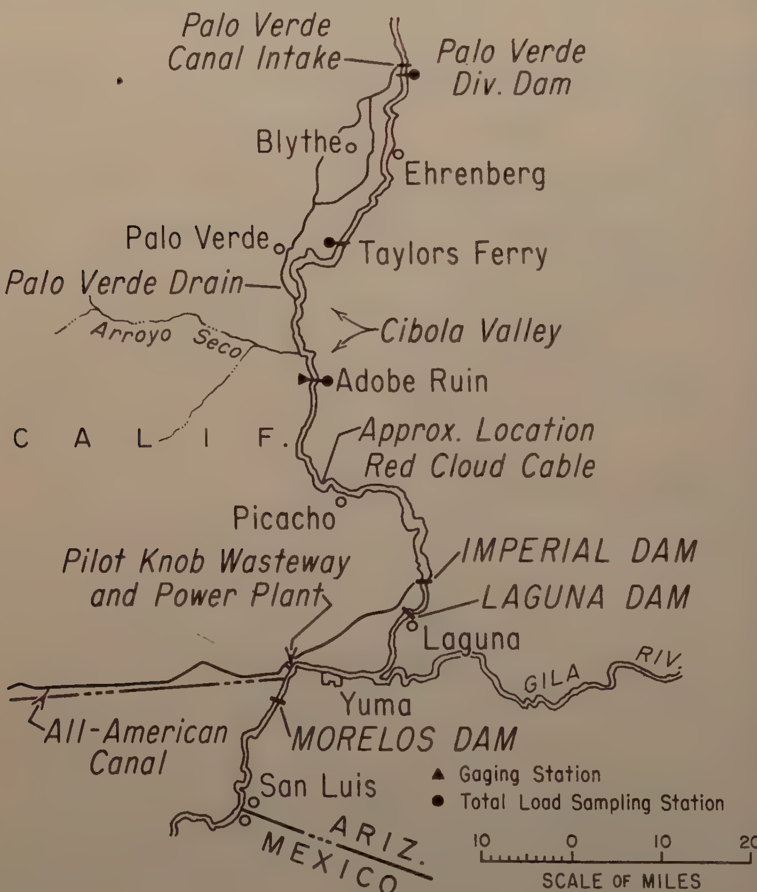

FIG. 3.—LOWER COLORADO RIVER, PALO VERDE DAM TO INTERNATIONAL BOUNDARY

computations are shown on Table 2. Average-size analysis of suspended and bed materials for the total load stations are shown in Fig. 5. Information obtained includes the sediment load at Needles, Calif., the inflow to Topock Gorge and Lake Havasu, and the movement into and through the Cibola Valley. The river cross sectioning gives information on general river aggradation and degradation, and the detailed sampling gives information on the movement pas

4 "Computations of Total Sediment Discharge, Niobrara River near Cody, Nebraska," U. S. Geol. Survey Water Supply, Paper No. 1357, 1955.

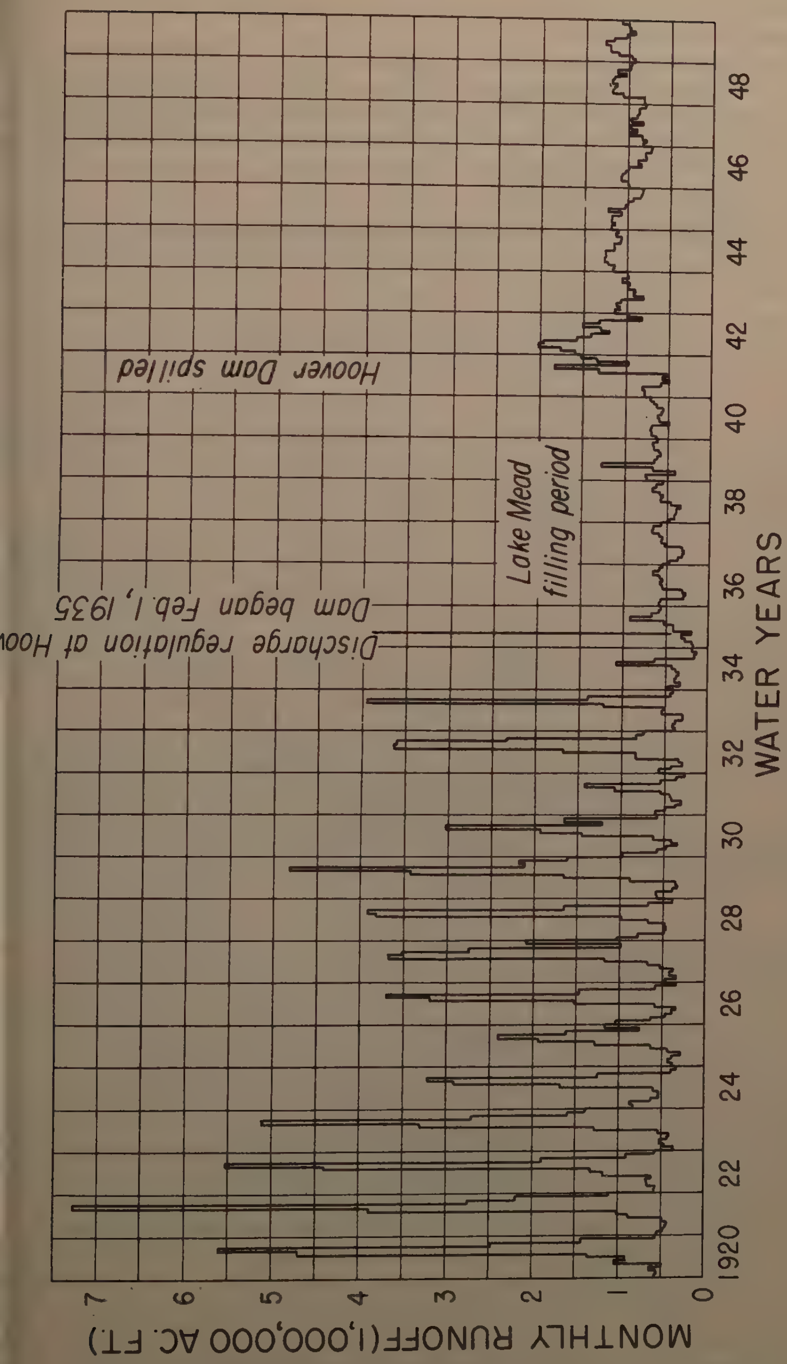

FIG. 4.—MONTHLY HYDROGRAPH OF FLOWS AT TOPOCK, ARIZ.

a specific point, including values that can be used in river shape-determination equations and relationships.

c. Bed material samples.—These samples have been collected in conjunction with the river cross sectioning and with the total transport-sampling measurements. Average bed-material-size distribution curves are shown in Fig. 5.

d. River discharge records.—These records are available at several strategic locations throughout the river reach involved. The gaging station locations are shown on Figs. 2 and 3.

For details on the sediment sampling, bed material sampling, river cross sectioning, and river discharge, reference is made to publications such as the

TABLE 2.—SUMMARY OF AVERAGE YEARLY TOTAL SEDIMENT LOAD SINCE CLOSURE OF DAVIS DAM[a,b]

River Reach	Station	Average Yearly Total Sediment Load (Tons)	Average Yearly Water Discharge (Acre Ft)[c]
Davis Dam to Parker Dam	R. S. 33	7,234,000	10,319,000
	Below Needles Bridge	6,804,000	10,319,000
	R. S. 41	4,503,000	10,319,000
	R. S. 43	4,791,000	10,319,000
Parker Dam to Imperial Dam	Below Palo Verde Weir	2,867,000	8,882,000
	Taylor's Ferry	3,646,000	8,882,000

[a] Based on flow-duration, sediment rating curve analysis. [b] Table 15 of "Interim Report, Total Sediment Transport Program, Lower Colorado River Basin," January 1958. [c] Davis to Parker reach based on flow-duration curve for U.S.G.S. station near Topock, Arizona, for calendar years 1950-56. Parker to Imperial reach based on flow-duration curve for U.S.B.R. station estimated daily flows at Taylor's Ferry for calendar years 1951-56.

U. S. Geological Survey Water supply papers, and reports on River Control Work and Investigations.[5,6]

SCOPE OF CURRENT RIVER RECTIFICATION

Until recent years, the river rectification work carried out was of an emergency nature to alleviate the immediate problems. Examples are the channelization below Needles in 1953, to lower the rising river level; channel cut-off in the Cibola Valley below Blythe, Calif., to lower river water surface at the Palo Verde Irrigation District outfall drain; and a cut-off above Yuma to move the flow of the river away from an air-strip that was being damaged by bank erosion. Current and proposed river channelization and control work is now more comprehensive in nature. Conditions between major channel structur

[5] "Report of River Control Work and Investigations, Lower Colorado River Basi U. S. Bur. of Reclamation, Boulder City, Nev., November, 1957.

[6] "Interim Report, Total Sediment Transport Program, Lower Colorado River Basi Sedimentation Sect., Hydrol. Branch, Bur. of Reclamation, Denver, Colo., January, 19

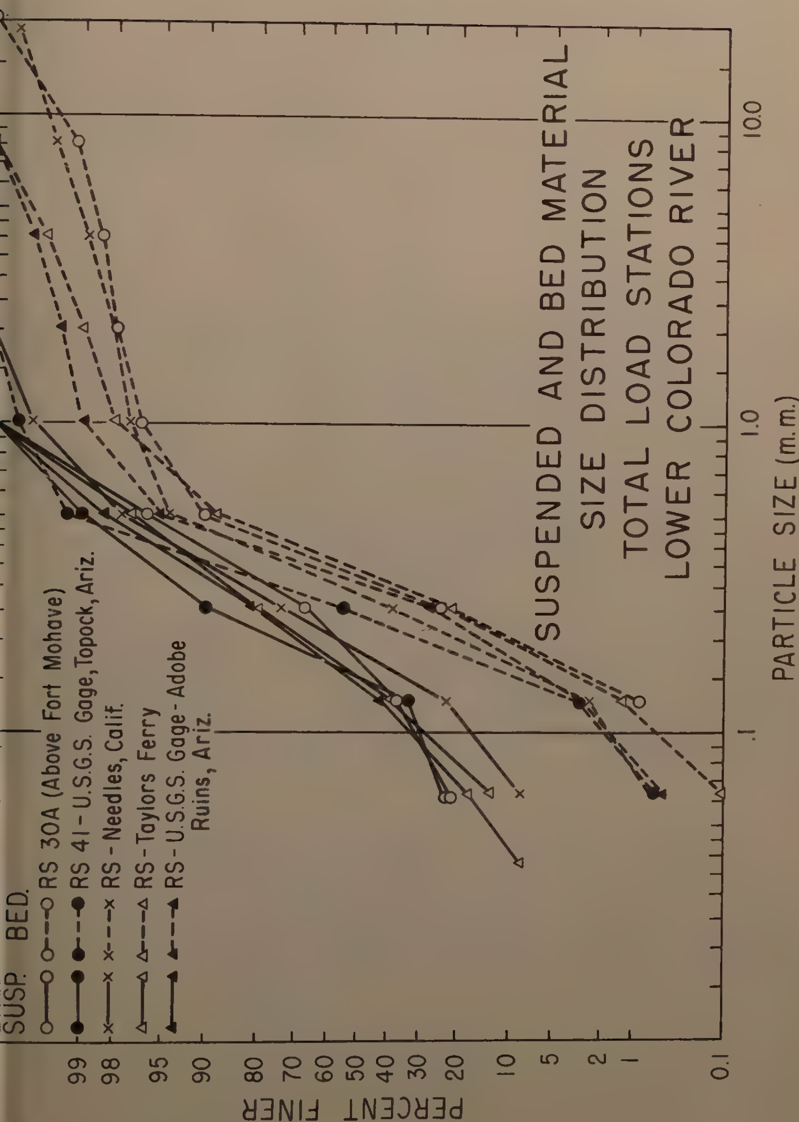
SUSPENDED AND BED MATERIAL
SIZE DISTRIBUTION
TOTAL LOAD STATIONS
LOWER COLORADO RIVER
SUSP. BED.
RS 30A (Above Fort Mohave)
RS 41 - U.S.G.S. Gage, Topock, Ariz.
RS - Needles, Calif.
RS - Taylors Ferry
RS - U.S.G.S. Gage - Adobe Ruins, Ariz.
PERCENT FINER
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.1
PARTICLE SIZE (m.m.)
.1
1.0
10.0

FIG. 5

are being analyzed. The work plan between these structures includes an evalu-ation of all current and future conditions in that river reach. There are three basic reaches involved in this comprehensive study approach:

a. Davis Dam to Parker Dam (Lake Havasu)
b. Parker Dam to Imperial Dam
c. Imperial Dam to the International Boundary

Note that the reach between Hoover Dam and Davis Dam (Lake Mohave) is not included, since Davis Dam backs water to Hoover Dam and the whole reach is in a canyon. For purpose of economic evaluation, the Parker to Imperial Dams reach has been broken in two subreaches (1) Parker to Palo Verde Dam (Fig. 2), and (2) Palo Verde to Imperial Dam (Fig. 3).

AGGRADATION AND DEGRADATION

The clear water released from the Hoover, Davis, and Parker Dams has resulted in scour of the riverbed and bank materials in the channel below the dams, with resultant aggradation in the backwater reaches of the next struc-ture downstream. General degradation and aggradation are illustrated in Fig. 6

This scour and fill of the river alluvium results in a lowering of water sur-face in one location and an increase in water surface in another location. I intermediate reaches an unstable, braided-type channel develops which allow the river to attack the vulnerable banks. Degradation introduces a continua supply of sediment to the downstream aggradation areas and results in deterio-ration of riverbanks and bed. Aggradation results in a rise in river-water sur-face and ground-water surface, growth of phreatophytes, and wide alluvial de-posit areas.

As has been discussed previously, the bulk of the sediment movement in th Lower Colorado River (since closure of Hoover Dam) is derived from the river bed and banks. This is important in the planning of river rectification wor because if this attack on the bed and banks can be brought under control, stable condition will eventually develop. This is the aim of the present rive control and rectification work, and planning program being carried out by th USBR.

RIVER CONTROL AND RECTIFICATION PROGRAM

Probably the best way to discuss the program for river control and rectifi-cation in the Lower Colorado River is to begin at Hoover Dam and work down-stream, examining each problem area in turn, pointing out the studies ar analysis made, the design considerations, construction program, and result obtained. The discussion is confined to the major problems and problem areas Many localized-type problems, associated with the operation and the river, ar continually developing within the major problem reaches.

Davis Dam to Lake Havasu.—With closure of Hoover Dam in 1935, and sub-sequent closure of Davis Dam in 1950, a degradation cycle began in the rive downstream. Much of the material scoured from the river channel deposite in the lower Mohave Valley near Needles (Fig. 2). This lower valley was his-torically an aggrading reach, caused in part by the constriction as the rive entered the Topock Gorge. The aggrading conditions were aggravated by th closure of Parker Dam in 1938, and subsequent filling of Lake Havasu, whic created a backwater extending through the Topock Gorge. The aggradation

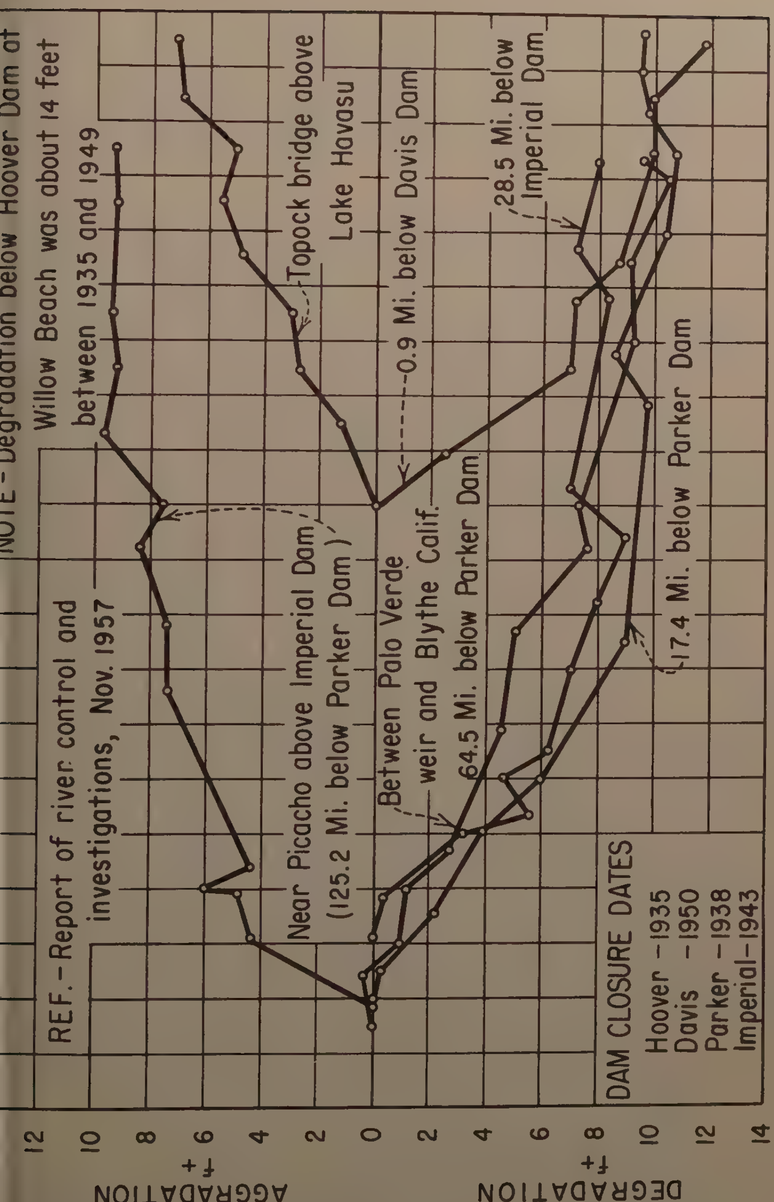

FIG. 6.—AGGRADATION - DEGRADATION AT VARIOUS LOCATIONS

the area above Topock raised the river-water surface elevation and ground-water elevation to a point that endangered the town of Needles. In 1946, emergency legislation by the Congress enabled the USBR to purchase a dredge for the purpose of constructing a channel that would lower the water surface in the Needles area. The dredge purchased was a 20-in. cutter-head type and has a design capacity of 500 cu yd per hr and can move 1,300 cu yd per hr under favorable conditions. The emergency channel constructed was 200 ft to 400 ft wide, and banks were riprapped as necessary. This channel was completed in 1953, and an immediate lowering in water surface of 5 ft was realized in the Needles area.

Although this emergency channel work alleviated the situation, it was realized that the effect would be only temporary because the source of materials, the broad Upper Mohave Valley, was still contributing material to the Needles-Topock reach at an unabated rate. Further, the supply of material in the Upper Mohave Valley, estimated at 112,000,000 cu yd, was sufficient to continue the sediment inflow to the lower valley for many years. The permanent solution became obvious: The river had to be confined in a controlled channel from Davis Dam to the mouth of Topock Gorge, a distance of about 43 river miles. Actually, because the river is confined in a well armored channel from Davis Dam to Big Bend, a distance of about 10 miles, the channelization length requirement was reduced to 33 river miles. To arrive at a comprehensive plan for a stable controlled channel, some specific information and data on existing conditions had to be obtained. Deep bed-material sampling was accomplished by use of a "sand-bucket" and/or auger which were driven 10 ft maximum depth or to refusal. Samples were retained for various depths and the variation in material size with depth was established. The total sediment loads above Needles, at Needles, and in the Topock Gorge were determined by use of the Modified Einstein Procedure.[4] A dominant discharge of 15,000 cfs was determined from an analysis of Hoover and Davis Dam releases and a flood potential of 50,000 cfs from Davis Dam to Piute Wash, and 70,000 cfs from Piute Wash to Topock was computed for use in levee design. Determination of the dominant discharge was complicated to some extent by the daily fluctuation in releases from Davis Dam, which is used for power-peaking purposes. The relatively high stage of Lake Havasu and its effect on the channel in the lower part of the reach was an additional factor considered in the design.

Since the sediment load in this reach of the river would, with channel control, decrease with time from the present amount of about 18,000 tons daily to a negligible amount, this factor had to be considered in the channel design. It is interesting to note that although the sediment concentration at Needles is only about 400 ppm (mostly fine to medium sand), the tonnage carried is high because of the high discharge. When the channel has been established at design width, and banks have been riprapped as necessary, vertical degradation within the controlled channel can be anticipated. This degradation will cause a sifting or sorting of the bed material until a sufficient layer of armoring material is established.

With the available data and the above factors in mind, the most compatible channel shape and alinement were established. Several available methods were used in the channel design including relationships and equations developed by Leopold and Maddock,[7] Schoklitsch,[8] and Lane.[9] The results of these relation-

[7] "The Hydraulic Geometry of Stream Channels and Some Physiographic Implications," by Luna B. Leopold and Thomas Maddock, Jr., Geol. Survey Professional Paper 252, 195-

[8] "The Scholklitsch Bedload Formula," by Samuel Shulits, Engineering, London, England, June 28, 1935.

[9] "Design of Stable Channels," by E. W. Lane, Transactions, ASCE, Vol. 120, 1955.

hips and equations as applied to the available data are shown on Fig. 7. Com-uted roughness values for the channel vary from 0.025 to 0.030. The amount f degradation that could be expected in the design channel width was deter-ined by use of the Schoklitsch bedload equation, competent bottom velocity ethod, and tractive force criteria as applied to the deep bed material samp-ing results. Through the process of sifting and sorting of the bed material, he transportable sizes are scoured and the general channel gradient is flat-ened. At a certain combination of bed material size, channel gradient, and

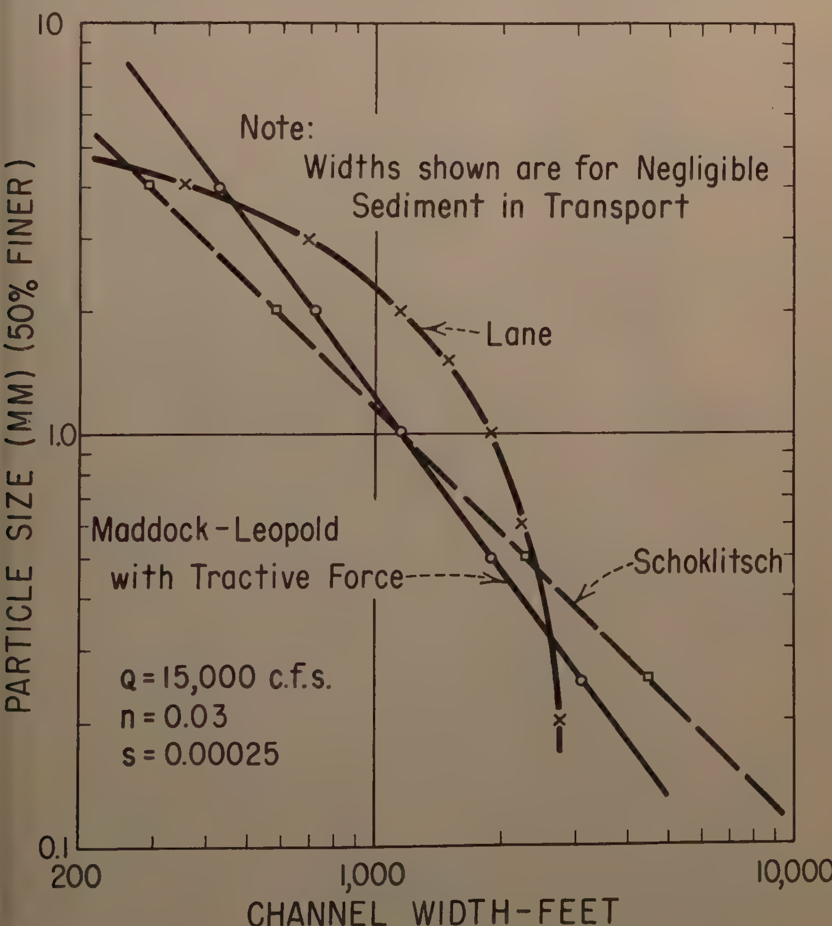

FIG. 7.—COLORADO RIVER CHANNEL STUDIES

her hydraulic factors, stability is reached. It was determined that bed sta-lity would occur when a 6-in. armor layer of 4 to 10 mm material was ob-ined. Scour of 7 to 10 ft was determined for the upper end of the controlled each reducing to a few feet near Needles (Fig. 8). The final channel design ape, profile, and alinement is shown in Fig. 9. The overall plan, including ork accomplished to date, involves the dredging of over 20,000,000 cu yd of aterial and placing of about 236,000 cu yd of riprap. The dredged channel

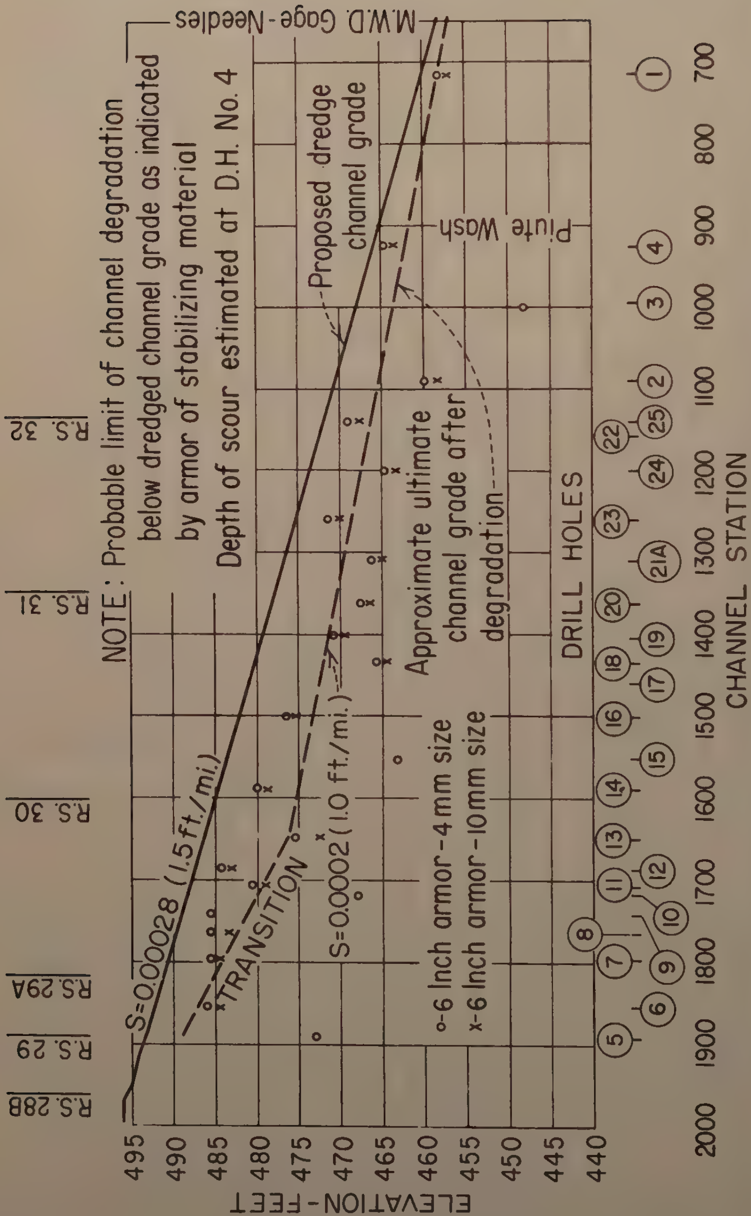

FIG. 8.—COLORADO RIVER CHANNELIZATION, BIG BEND TO NEEDLES

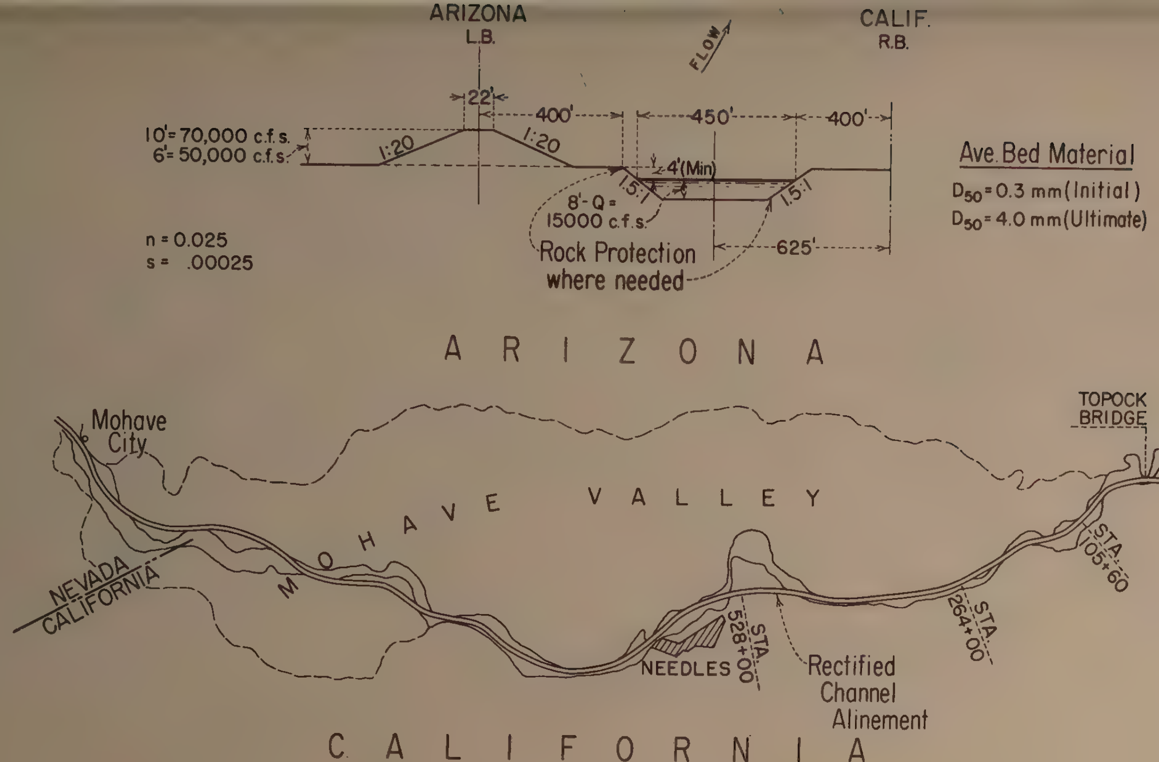

FIG. 9.—COLORADO RIVER CHANNELIZATION, BIG BEND TO TOPOCK

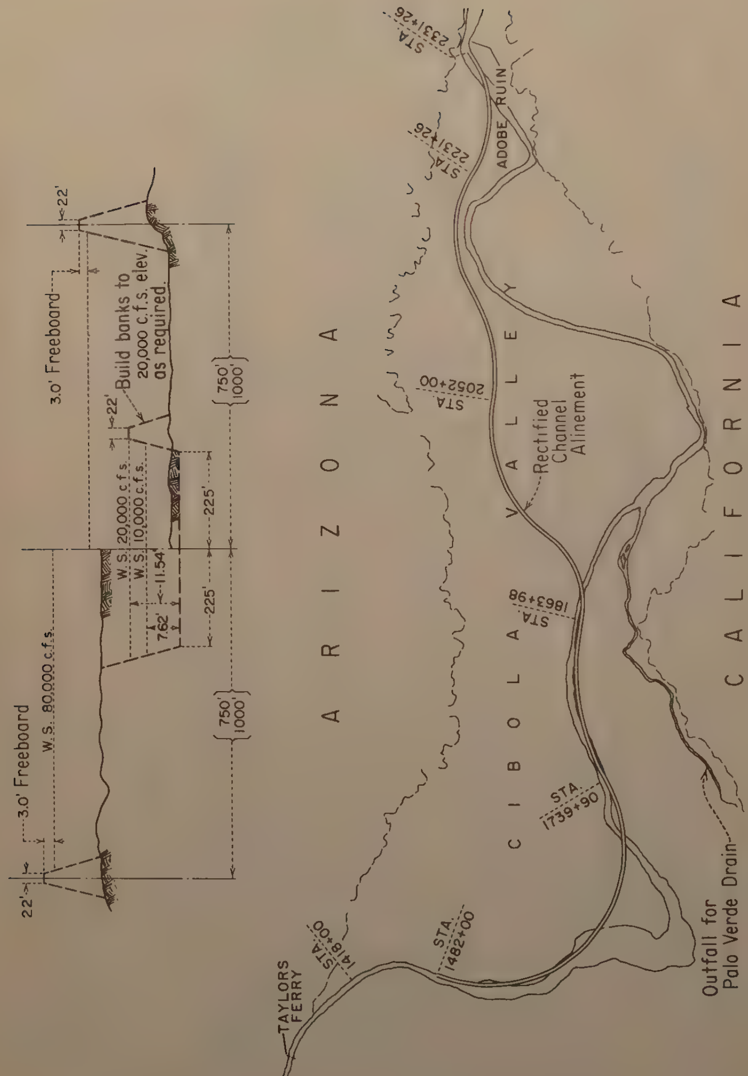

FIG. 10.—PROPOSED COLORADO RIVER CHANNELIZATION, TAYLORS FERRY TO ADOBE RUN

ength is about 30 miles, and is on a sinuous alinement with an average bend adius of 10,000 ft. Radii limitations are 5,700 ft minimum and 16,000 ft maxi-um.

Construction of the design channel has now progressed to the lower end of ig Bend. The dredge is working upstream on the established alinement with edge material being used for levee building as required. A channel width of

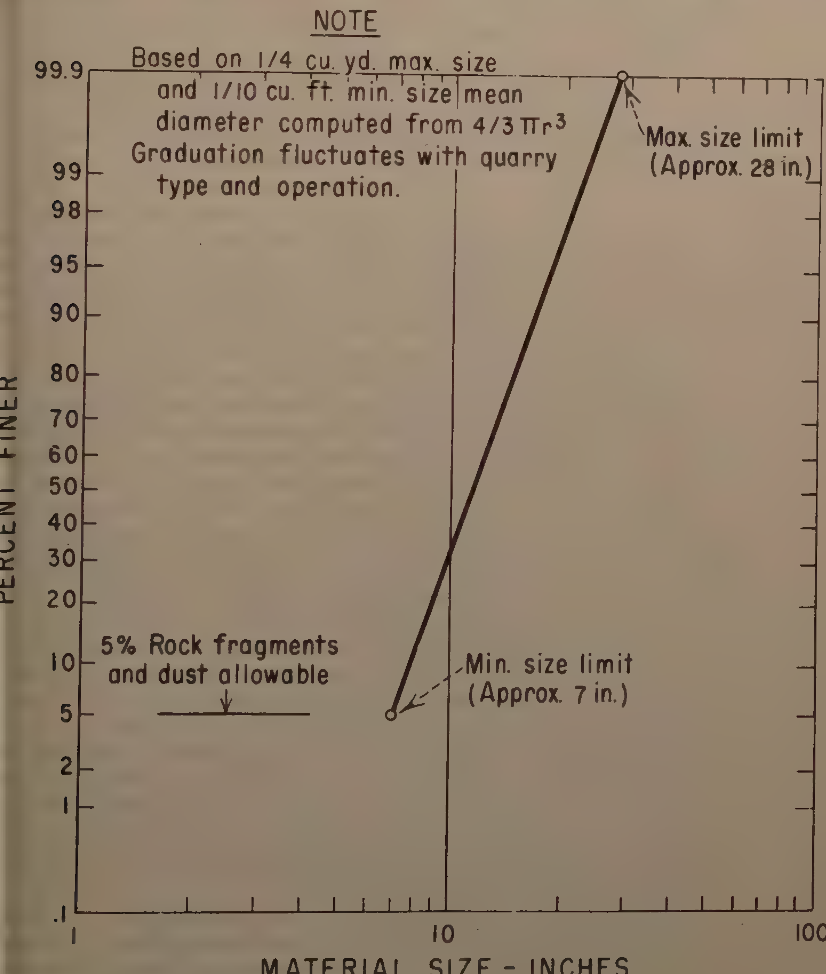

FIG. 11.—APPROXIMATE GRADATION RIPRAP, BIG BEND TO NEEDLES

to 400 ft is cut depending upon the amount of material needed for levee con-uction. One pass of the dredge will create a channel 170 ft wide. Additional ening to design width is accomplished by river action. Riprap is placed on outside of the bends only, except in reaches with large radii or where other ditions warrant riprapping on both banks. Generally, the riprap is placed he bank at design width and it falls in place as the channel widens. Riprap dation is shown in Fig. 11. The dredge is moving material at the rate of

14,000 cu yd daily, working three shifts. Channel completion is scheduled fo early 1960. Recently, the proposed termination point for the channelization wa moved downstream a short distance to accommodate objections of local proper ty owners in the vicinity of Bullhead City.

Because the sediment inflow to the Needles area will continue for man years, though at a decreasing rate, it was deemed advisable to acquire a small er dredge to maintain the Needles to Topock channel while the upstream chan nel was being completed, and to maintain the completed channel in future years A 12-in. cutter-head dredge, with a capacity of 300 cu yd per hr, was pur chased, in 1957, for this purpose and is currently operating in the area abov Topock Gorge. This has resulted in some stabilization of the river stage in th gorge and the water surface is being maintained fairly constant at Needles. greater length of time will be required to determine the full results of thi program.

During 1958, above-normal, sustained, high flows (20,000 to 25,000 cfs) wer experienced in the Davis to Lake Havasu reach. The designed channel, wher completed, functioned very well under these high flows. It is anticipated th the most troublesome factors, under future operation, will probably be grav and cobble bars extending into the channel because of flash floods on tributar arroyos, and bar building at cross-overs within the sinuous channel.

A resurvey of the active capacity of Lake Havasu was carried out in 195 and results indicate 58,100 acre-ft of capacity has been lost to sediment be tween operating elevations of 430 and 450. This represents 15% of the origin capacity within these elevations. This loss is primarily due to delta develop ment at the upper end of the reservoir. Water is pumped from Lake Havas into the Colorado River Aqueduct for the Metropolitan Water District.

With completion of the Davis to Topock channelization, a rapid reducti in the sediment transport of the river will be realized, thereby decreasing t sediment inflow to Lake Havasu to a negligible amount and creating a controll water surface in the river at Needles. In addition to these benefits from t channelization work, a significant amount of water salvage is accomplished reducing water loss to phreatophytes and evaporation, since the wide shall aggradational reaches are eliminated. Flood protection from levee constructi and enhancement of recreational facilities are additional benefits.

Data on river conditions, in the Davis Dam to Lake Havasu reach, will co tinue to be collected to aid in evaluation of the design plan. When the riv channel has become stabilized, these data will aid in evaluating the various d sign equations which were used. Total sediment, load measurements and di charge records will help to evaluate the effectiveness of the plan in reduci the sediment load and increasing water salvage.

Parker Dam to Palo Verde Diversion Dam.—No critical channel proble have developed between Parker Dam and Palo Verde Diversion Dam—a distan of 65 river miles (Fig. 2). However, river cross sectioning and total sedime transport determinations indicate that a considerable tonnage of sediment being derived from this reach. Preliminary investigations have been carri out to determine the type of surface and deep bed material, and general riv channel conditions. Indications are that no immediate channel rectificati work is necessary, but it is evident that because of the sediment contributed this stretch of the river to downstream reaches, future work will probably b come necessary. Headgate Rock Diversion Dam, which supplies water to Ind lands, is located about 15 miles below Parker Dam.

The first major problem to develop below Parker Dam was at the headworks of the Palo Verde irrigation works which is located about 10 miles upstream from the city of Blythe, Calif. Riverbed scour at the canal headworks continued until diversions could not be satisfactorily made. This lowering of the water surface was arrested in 1945, by the construction of a temporary rock weir. Maintaining this weir proved to be a continuous problem. New rock had to be added to replace rock moved on downstream during high river stages. Approximately 190,000 cu yd of rock were placed in the temporary weir between 1945 and 1956. Authorization was made by Congress, in 1954, for construction of a permanent structure a short distance downstream from the temporary weir. This structure, the Palo Verde Diversion Dam, was completed and was in operation during the 1958 irrigation season. The temporary rock weir was modified in accordance with a requirement contained in the Congressional authorization for the dam. This requirement, for nullifying the effects of the temporary weir, involved some detailed study to determine what cross sectional shape should be established, and how much rock removal was required to obtain this section. Removal of the rock also constituted a unique problem since it involved removing material of 1 to 3 cu yd and smaller while the river discharge was about 7,000 cfs. The work was accomplished satisfactorily by the contractor, however.

Associated with the scour problem at the Palo Verde Diversion Headworks was a sediment in-flow problem. Sediment diversion to the Palo Verde Canal has been heavy over the years—even before construction of upstream storage structures. A desilting basin and dredge have been maintained by the Irrigation District just below the canal heading. Because the new diversion dam gates and canal headworks can now be manipulated to minimize the sediment intake to the canal, the desilting operations can be curtailed.

Studies made in connection with the permanent diversion structure included water-surface profiles, upstream and downstream, and expected degradation in the downstream channel. Degradation was determined by extending the slope trend that was occurring to an ultimate river slope of 1.0 ft per mile. It was estimated that the bed would stabilize at the slope of 1.0 ft per mile. The upstream, water-surface profiles were required to determine levee elevation along the Indian lands on the east side of the river. The downstream profiles and degradation studies were required for tail-water determinations and structure design. Ultimate channel degradation of 8 ft, beyond that existing at time of construction of the new structure, is expected.

Palo Verde Diversion Dam to Imperial Diversion Dam.—The river channel problems in this reach of the Lower Colorado River are currently being investigated and studied in detail by the USBR. Problems include bank erosion and channel deterioration between Palo Verde Diversion Dam and Ehrenburg Bridge, near Blythe; channel aggradation with accompanying drainage, water loss, and channel capacity problems in the Cibola Valley, which extends from below Taylor's Ferry to the headwaters of Imperial Reservoir, and aggradation and sediment inflow to Imperial Diversion Dam and headworks. An approximate balance point between aggradation and degradation occurs near the Palo Verde outfall drain in the Cibola Valley. Between Ehrenburg Bridge and Taylor's Ferry, the river is in a relatively stable reach which holds against the east side bluff and has a surface gravel bed in several locations. Fig. 3 shows the location of these features.

Current planning for this stretch of the river involves channel straightening and bank protection in the upper reach and construction of a new channel in

the Cibola Valley. This channel-rectification plan is shown on Fig. 10. In the upper reach, some of the local land owners are placing protective works to stop bank scour. The primary objective of rectification in the Cibola Valley is to create a stable, controlled, river channel that will require a minimum of future maintenance. Other benefits, such as losses to nonproductive phreatophytes, lowering of the water surface at Palo Verde outfall drain, and recreational and fish and wildlife improvement may result incident to this work.

Severe drainage problems were reported in the lower Palo Verde Irrigation District lands because of the aggrading type channel in the Cibola Valley. As was the case upstream at Topock, there is a natural constriction of the river at the lower end of the Cibola Valley, near Adobe Ruin (Fig. 10). In 1947, a pilot cut-off was made just upstream from the outfall drain to ease the drainage situation. This cut was $1\frac{1}{2}$ miles long and 40 ft wide, and was accomplished by dragline operation. Additional widening was by river action. The work resulted in a lowering of water surface of 1 ft at the mouth of the out-fall drain. However, poor drainage and channel deterioration continued. It was concluded that major rectification in the Cibola Valley was needed. The first plan for such rectification was prepared in 1951.[10] Since then, data acquisition has been carried out to aid in correctly evaluating the existing conditions and to aid in formation of a comprehensive plan for rectification.

One of the most interesting and challenging aspects of the river planning and design in the Cibola Valley, is the proposed cut-off of about 9 miles in length from Station 1863+98 to Adobe Ruins (Fig. 10). A cut-off in the alluvial Cibola Valley deposits must be designed to remain stable with minimum scour upstream and minimum aggradation downstream under existing and future sediment inflow and dominant channel discharge. A dominant discharge of 10,000 cfs has been established from analysis of past and probable future river operation. The sediment inflow to the reach is 10,000 tons daily, as determined from Modified Einstein computations at Taylor's Ferry. The gravel bed elevation below Taylor's Ferry and the estimated maximum bed aggradation elevation at Adobe Ruin at the lower end of Cibola Valley must be met.

For the existing bed material and sediment load, it was determined that a channel gradient of 1.2 to 1.3 ft per mile should be used to obtain stability. Since the straight-line slope between stations would be considerably more than the 1.2 to 1.3 ft per mile limit, it was evident that a sinuous type of cut-off channel had to be constructed to meet the slope limitations. Previous experience by the USBR, the Corps of Engineers,[11] and others, indicates that the bend radii should not be less than 5,000 ft for the hydraulic conditions in the sinuous channel.

The design width for the new channel in the Cibola Valley has been tentatively determined as 450 ft maximum, by applying the acquired data and information to the Maddock-Leopold[7] and tractive force[9] relationships. The results were checked against data from actual field measurements at selected stable reaches in the valley and agreed very well. As in the Needles area, the outside of the bends of the constructed sinuous channel would be riprapped. It is interesting to note that unlike the channelization in the Topock to Davis Dam

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10 "Report on Colorado River Channelization, Cibola Valley, Arizona," U. S. Bur. of Reclamation, Colo. River Front Work and Levee System, Report No. RC-3-3.1, Boulder City, Nev., March, 1951.

11 "A Laboratory Study of the Meandering of Alluvial Rivers," by J. F. Friedkin, U. S. Waterways Experiment Sta., U. S. Corps of Engineers, Vicksburg, Miss., May, 1945

each, there will be a continuous sediment inflow to the Cibola Valley until hannelization is carried out in the Parker Dam to Palo Verde Diversion Dam each, (Fig. 2) which presently is not planned. A report is currently in prepa-ation which will present a comprehensive plan, and show benefits to be derived, y river rectification in the river reach between Palo Verde Diversion Dam nd Imperial Reservoir.

Imperial Dam and Reservoir.—One of the most widely known and referred desilting operations is the one at the intake to the All-American Canal at nperial Diversion Dam (Fig. 3). At the right side of Imperial Dam, the ma-ority of the river flow is diverted into the All-American Canal which has an nitial capacity of 15,155 cfs. Water is also diverted at the left side of the dam the Gila Gravity Main Canal which has an initial capacity of 2,200 cfs. Water assing the dam is for purposes of river regulation, which is discussed subse-uently, and for Mexican Treaty requirements. At the time of construction of nperial Dam, it was realized that large tonnages of sediment, primarily sand, ould be taken into the All-American Canal. Detailed studies, including labo-atory models were undertaken to devise a desilting works that would remove e sediment and return it to the river below the dam. The final desilting work lan consisted of six settling basins, arranged in pairs, equipped with twelve otating vanes or scrapers per basin that flushed the deposited material, as it ettled, into sludge pipes. The sludge pipes dumped into the Imperial Dam luiceway channel.[12] To date, the desilting has been completely satisfactory, s illustrated in Fig. 12. Sediment concentrations at Station 60 on the All-merican Canal are a very small fraction of the concentrations entering the esilting basins.

The reservoir above Imperial Dam, at the time of closure in 1938, had a ca-acity of 85,000 acre-ft. The reservoir filled with sediment at a rapid rate ereby reducing the reservoir trap efficiency and allowing more sediment to each the All-American Canal headworks. The present reservoir capacity is nly 1,000 acre-ft.

The sediment inflow will be reduced to some extent by bank protective works the Palo Verde area and river rectification in the Cibola Valley but will still e a problem at Imperial Dam.

In addition to the desilting works, sluicing operations are carried out peri-dically at Imperial Dam to remove sediment deposits upstream from the All-merican Canal headworks structure. Detailed sediment sampling and dis-harge measurements are made during these sluices to aid in evaluating the peration, and for use in planning downstream river regulation. A desilting nd sluice arrangement is also provided at the Gila Gravity Main Canal head-orks, but because of the relatively small amount of water diverted, the sedi-ent problem is minor at this point.

The sediment problem will continue at Imperial Dam for many years to ome because it is the terminal point for material carried down the river from pstream sources. Data collection and study will be continued by the USBR, to eep a check on conditions and to assist in planning of any future rectification ork, should such work become necessary.

Imperial Dam to International Boundary.—Below Imperial Dam, the major roblem, at present, is that of river regulation. River regulation, as referred here, is the maintaining of a suitable channel that will transport the sedi-ents introduced at Imperial Dam without creating channel deterioration. This

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12 "Imperial Dam and Desilting Works," Bulletin No. 6, Boulder Canyon Proj. Final ports, Part IV, Design and Constr., U. S. Bur. of Reclamation, Denver, Colo., 1949.

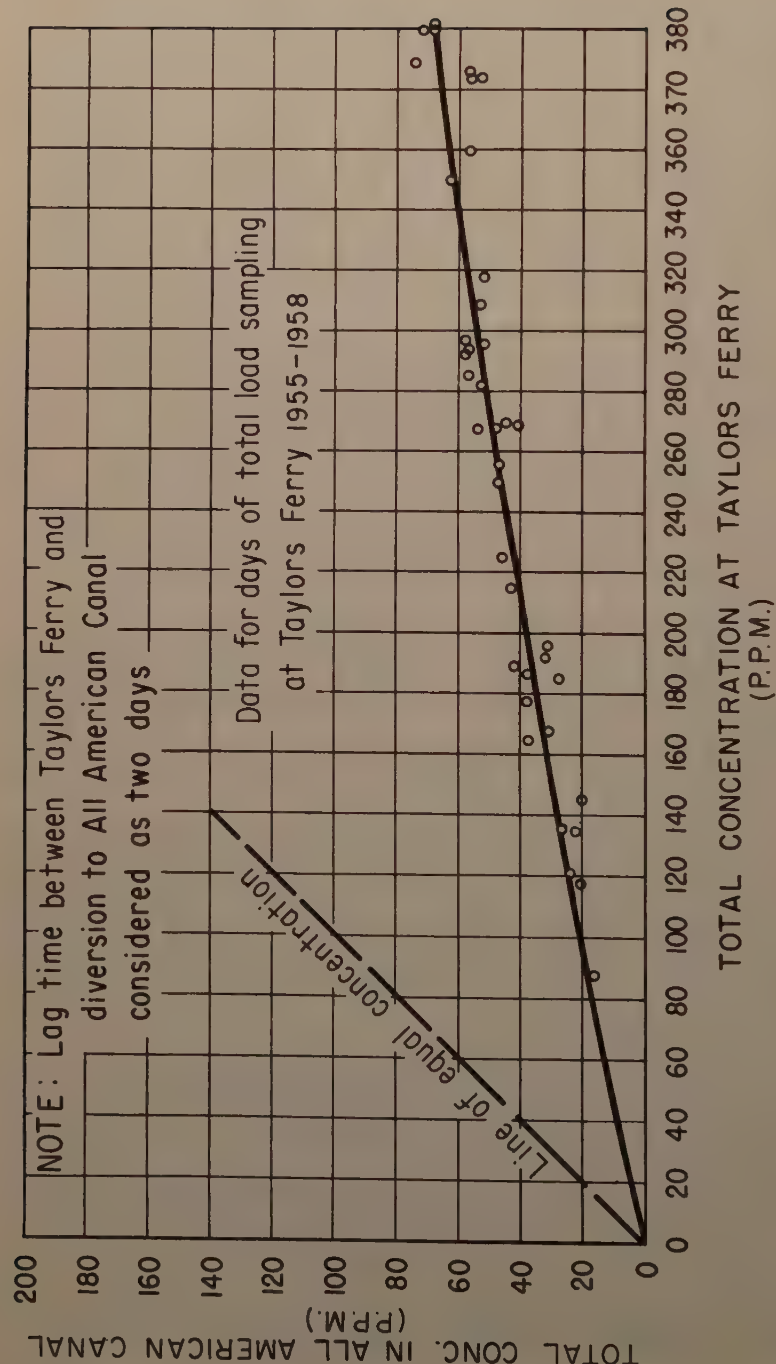

FIG. 12.—RELATIONSHIP BETWEEN TOTAL SEDIMENT CONCENTRATION FOR INFLOW TO IMPERIAL DAM AT STATION 60 ON ALL-AMERICAN CANAL

; accomplished by the release to the channel downstream of a certain per-entage of the water flowing into Imperial Reservoir. Water is released at oover Dam in accordance with a weekly master schedule based on require-ients of all water users. This schedule can be fluctuated within set limits to ieet varying demands throughout the year. Based on studies of sediment data ollected at Imperial Dam and at Yuma, some relationships have been estab-shed for use in predicting the amount of water needed for river regulation. ypical relationships are shown in Fig. 13. These curves are periodically re-ised to reflect latest data. The water used for river regulation can be the ame water released from Imperial Dam to meet Mexican Treaty requirements. 'hen the Imperial Irrigation District so requests, the water not required for ver regulation is put into the All-American Canal and returned to the river : the Pilot Knob Powerplant (Fig. 3).

With the initial closure of Imperial Dam, degradation occurred downstream. his scouring action reverted to aggradation as Imperial Reservoir filled with ediment. Generally, the river channel has not yet recovered to the condition hich existed upon the Imperial Dam closure. With continued demands on the olorado River water and development of the water uses above Hoover Dam, e water available for river regulation below Imperial Dam will become less, id maintaining a suitable river channel will become more difficult. For this eason, new methods of silt control may become necessary. This situation was redicted in the studies and analyses made of the situation during the planning iases for Imperial Dam and desilting works.[12]

It is interesting to note that the total sediment load now passing the Yuma ging station is only a fraction of that which passed the same point prior to nstruction of Imperial and Hoover Dams (Fig. 14).

About 5 miles below Imperial Dam, there is an old irrigation diversion ructure, Laguna Dam, which was built in 1909 (Fig. 3). The only purpose is structure serves, at present, is to provide reregulation of releases from iperial Dam. Considerable sediment deposition has taken place in the wide ver area between the Imperial and Laguna structures.

Below Laguna Dam, the interests of the USBR and those of the International undary and Water Commission are closely allied. Therefore, the problems the reach from Laguna Dam to San Luis, Mexico, which is the lower boun-ry in the limitrophe reach, will not be discussed in any detail. The USBR's rthest downstream river work has been the rehabilitation of the existing levee stem above and below Yuma.[13] This levee rehabilitation was necessary to ovide adequate protection against probable floods and any increase in water rface resulting from construction of Morelos Diversion Dam, below Yuma, Mexico. Part of the expense of this levee rehabilitation was borne by Mexico. e work consisted of placing 1,700,000 cu yd of levee embankment, including rap, as necessary over a total length of 41 miles. The levees were designed a flood of 140,000 cfs. This flood value was determined considering Painted ck Dam would control much of the Gila River flood potential. Riprap having sual thickness of 5 ft was placed at critical locations. Riprap limitations re a maximum size of 1/2 cu yd and a minimum of 1/10 cu ft in volume.

Two interesting problems near the city of Yuma have resulted in rectifi-ion work by the USBR. At Yuma, another natural constriction of the Colo-lo River exists (Fig. 3). Here, the river narrows from a wide valley into a rge-type opening about 300 ft in width. The area above this constriction is

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13 "Brief History of Colorado River Levees Near Yuma, Arizona," U. S. Dept. of rior, U. S. Bur. of Reclamation, Boulder City, Nev., November, 1955.

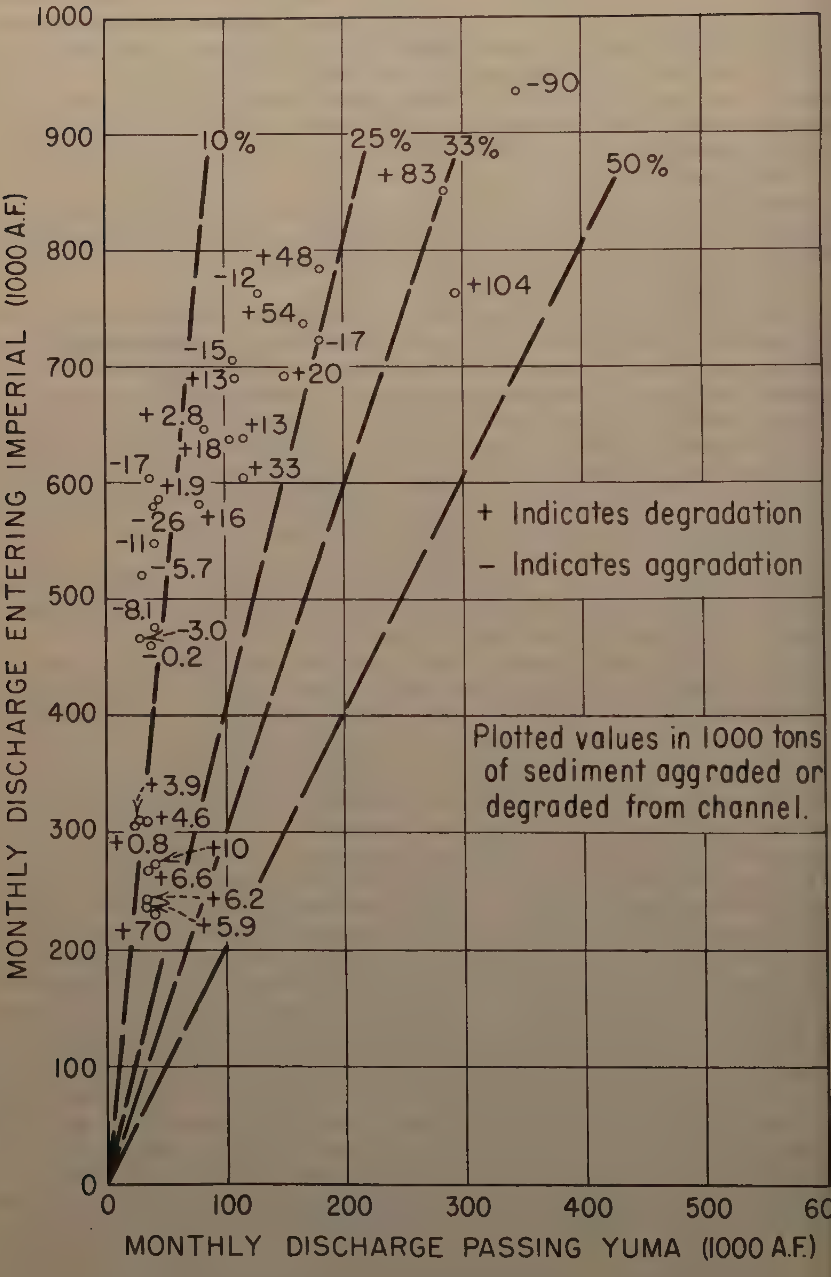

FIG. 13.—DATA FROM SLUICING MEASUREMENTS AT IMPERIAL DAM AN[D] AT YUMA

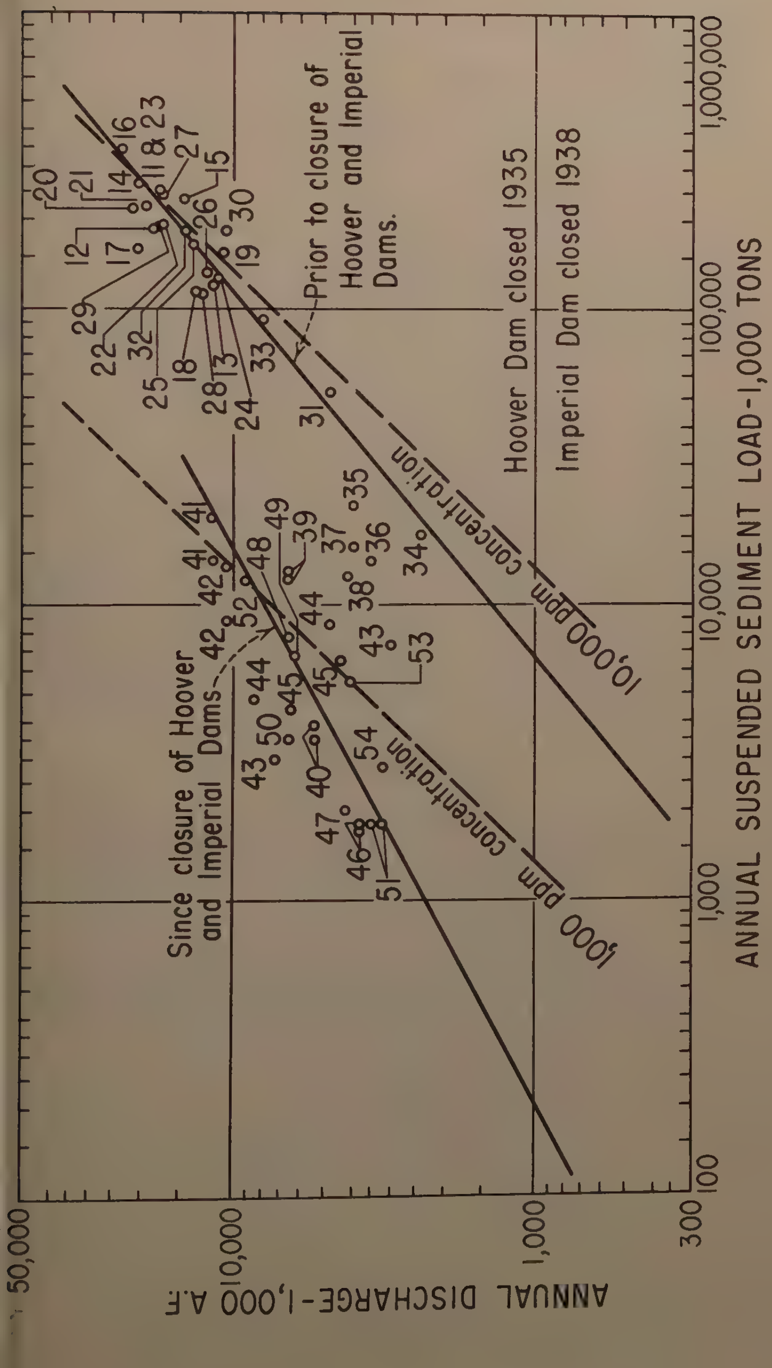

FIG. 14.—ANNUAL SUSPENDED SEDIMENT LOAD, YUMA GAGING STATION

a natural depository for sediment, particularly during high flows when a back-water effect is created. The river course through this depositional area is subject to considerable shifting. The river channel in the early 1950's moved in against the left bank, attacking existing low-lying airport lands immediately north of Yuma. To correct this condition, a pilot cut-off was constructed to change the river course and give it a direct approach to the constricted section. This cut-off involved excavating 200,000 cu yd of material, placing riprap at the upstream end, and constructing a structure across the old channel to obstruct flow and force the river into the pilot cut. Shortly after completion of the cut-off in 1954, the entire river was flowing through the new channel. This relocated section of channel is continuing to function satisfactorily.

Just below the constriction near Yuma, part of the city water supply is pumped from the left bank of the river. During low river flows, considerable difficulty has been experienced in maintaining an open channel from which the water could be pumped. This difficulty resulted from the manner in which the low flow channel entered the constricted reach and from bar buildup where the water treatment plant effluent is discharged into the river. To alleviate this situation, some deflection fencing was placed by the Bureau in 1957, under special appropriation by Congress. Immediate improvement was noted, and the severity of the problem was reduced during 1958, by the relatively high flows passing the Yuma area.

Requirements for river rectification and control work in the Imperial Dam to International Boundary reach are likely to continue for many years to come. Collection and analysis of data will continue, and a constant vigil will be kept to foresee the development of critical conditions and thereby provide ample opportunity to take necessary action. Exchange of data and cooperation in analyzing conditions will continue with the International Boundary and Water Commission as has been done in the past.

SUMMARY

The construction of Hoover Dam and other structures on the Lower Colorad
River has essentially solved the flood problems, created a firm irrigation wate
supply, and provided a large block of hydroelectric power. However, thes
structures did not eliminate the problems of river control, but only change
their character. The channel aggradation and degradation that have occurre
with the construction of water diversion and storage facilities have created
series of varied river control problems that require rectification. These rive
alterations are continuing and rectification needs will continue as long
sediment-contributing reaches of the river remain uncontrolled.

Rectification work, and planning for such work, is in various stages of de
velopment. In the river reach between Davis Dam and Topock, channel dredgi
is underway to create a new fully controlled channel. Work in this reach is 85
completed (as of 1960). The channel has been designed to be compatible wi
existing and future sediment load, bed material, discharge, and hydraulic fa
tors. In the Palo Verde Diversion Dam to Imperial Diversion Dam reac
studies are currently underway and a report will be made soon presenting
plan for river rectification. The plan will probably involve bank control, cha
nel straightening and realinement, and a substantial cut-off channel in the C
bola Valley. Downstream from Imperial Dam, sediment and hydraulic data a
being collected, and studies and analyses are being carried out for the purpo
of determining river regulation water requirements and to provide a consta

eck on river channel developments. Various items of river rectification ork are carried out as required within these major river reaches to correct localized channel problem.

Sediment movement and hydraulic data are collected on a regular basis at rategic locations throughout the Lower Colorado River Basin. Results and alysis of these data are published by the USBR.[5,6] Total sediment transport mpling stations are established or dropped as required by the detailed study ogram.

Since the river channel aggradation and degradation results from a shuffling d reshuffling of riverbed and bank materials above and below major strucres, planning and design for river rectification is aimed at obtaining eventual mplete control between these major control points. Since any work must be stified on the basis of benefits to be received, it is not possible to carry out a mplete construction program between major structures, but, rather, work ıst be concentrated on reaches or sections where benefits will justify the ctification costs. This intermediate type work is planned as an integral part the overall river control program, however.

River rectification work generally involves channel straightening and renement, utilizing cut-offs, riprap placement, training structures, and levees. ıe method used, in each case, is selected on the basis of existing and probable ure conditions. Improvements in methods of control are constantly being ade from the experience gained on work accomplished so far, and the analy- s of newly collected sediment, hydraulic, and hydrologic data. Present inditions are that future needs for river rectification will be in the Palo Verde version Dam to International Boundary reach. The most urgent need at esent is in the Cibola Valley.

Looking into the future, it is conceivable that the Colorado River will evenılly become fully controlled from Hoover Dam to the International Boundary. th increased population trends in this region of the United States, it can be pected that the water resource, including the recreational potential along ese 300 miles of river, will be utilized at an ever-increasing rate. A fully ntrolled river will allow the most complete use of the available water for the mbined benefit of all interests in the Lower Colorado River Basin.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to A. L. Mitchell, River Conl Engineer, Boulder City, Nevada, and Paul Oliver, Project Manager, Needles, lif., both of the USBR, for their cooperation and contributions to this paper.

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CONTENTS

DISCUSSION

FRICTION FACTORS IN CORRUGATED METAL PIPE[a]

Discussion by W. O. Ree

W. O. REE,[1] F. ASCE.—Data presented in this paper provided an oppor-ity for the writer to test a relationship he had used to estimate Manning's values for paved invert corrugated pipe. In 1953 the writer tested a full-ated, paved-invert, corrugated pipe of 18-in. diameter. Tests were made various depths of channel flow as well as for the full-pipe condition. Fric-n factors were obtained for this pipe but the problem of estimating n-values other sizes of paved invert corrugated pipe still remained. It was beyond ilable resources to test other sizes so resort was made to an estimating eme adapted from a method devised for open channels by Robert E. Hor-.[2]

Manning's formula was used in this analysis. The author's symbols will be ployed in the discussion plus a few additional ones. These are:

P = wetted perimeter, in feet;
p = subscript indicated paved; and
c = subscript indicating unpaved (corrugated).

nning's formula is:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

stituting $\frac{A}{P}$ for R and rearranging

$$\left(\frac{1.486\ S^{1/2}}{V}\right)^{3/2} = n^{3/2}\frac{P}{A}$$

pair of values n_p and n_c will be assumed to satisfy this equation, thus:

$$\left(\frac{1.486\ S^{1/2}}{V}\right)^{3/2} = n_p^{3/2}\frac{P_p}{A} + n_c^{3/2}\frac{P_c}{A}$$

September, 1959, by Marvin J. Webster and Laurence R. Metcalf.
Proj. Supervisor, Hydr. Lab., Agric. Research Service, USDA, in cooperation with Okla. Agric. Experiment Sta., Stillwater, Okla.
"Separate Roughness Coefficients for Channel Bottom and Sides," by Robert E. on, Engineering News-Record, Vol. III, November 30, 1933, pp. 652-653.

equating the equivalents,

$$n^{3/2}\,\frac{P}{A} \doteq \frac{n_p^{3/2}\,P_p}{A} + \frac{n_c^{3/2}\,P_c}{A}$$

solving for n,

$$n = \left(\frac{n_p^{3/2}\,P_p + n_c^{3/2}\,P_c}{P}\right)^{2/3} \quad \ldots\ldots\ldots\ldots\ldots \quad ($$

This is the prediction equation. The data from the writer's experime were used to test it. The experiment will be described briefly and the pert nent data tabulated.

The pipe was part of a full-scale pipe outlet spillway set up for testing the Stillwater Hydraulic Laboratory of the Agricultural Research Service a the Oklahoma Agricultural Experiment Station. The pipe was new, corrugate full coated, and had a paved invert covering 38% of the perimeter measured the minimum diameter. The test section was 200 ft long and was installed a 3.57% slope. Depths of flow were measured at 20-ft stations by open colur glass manometers connected to piezometer openings in the invert of the pip Twenty-six channel flows were run with depths ranging from 0.17 to 1.30 Velocities were high and depths were small so fluctuation and variation we experienced in the depth determinations. The depth measurements were ave aged for the ten stations and this smoothed the data. Even so, the sixteen te flows contained within the paved portion of the pipe yielded values of Manning n for the paving ranging from 0.0085 to 0.0112. Casting out the three extren values reduced the range from 0.0090 to 0.0099 with a mean value of 0.009 The ten remaining flows, which were deeper, yielded Manning's n-values whi showed a more consistent behavior. The hydraulic elements for these tes are given in Table 1.

The Manning's n-values in Table R1 are plotted against the correspondi depth values on Fig. R1(a). On this same figure is shown the calculated n-dep relationship. The agreement between the estimated and actual values of Ma ning's n is fairly good, being very good at full pipe flow. This agreement ga some encouragement as to the usefulness of Eq. 1. However, the writer w unwilling to give credence to the formula, because of both the nature of its or gin and the quality of the data used in the proof. One uncertainty factor, hithe to unmentioned, was the value of n_c. No data were available on coated pipe the value of 0.025 for plain corrugated pipe reported by Straub[3] was used in estimate of the composite n. Another matter for concern was that the te were in the super-critical flow range. In view of these uncertainties the id was allowed to lie dormant until the data reported by Webster and Metcalf we used to test the prediction equation.

One difficulty in the application of the Webster-Metcalf data was the lack an experimental determination of n_p (Manning's n for the paving alone). A val of n_p was calculated by applying Eq. 1 to the lowest flow in the 5-ft diamet 25% paved invert pipe. The result of this determination agreed exactly with

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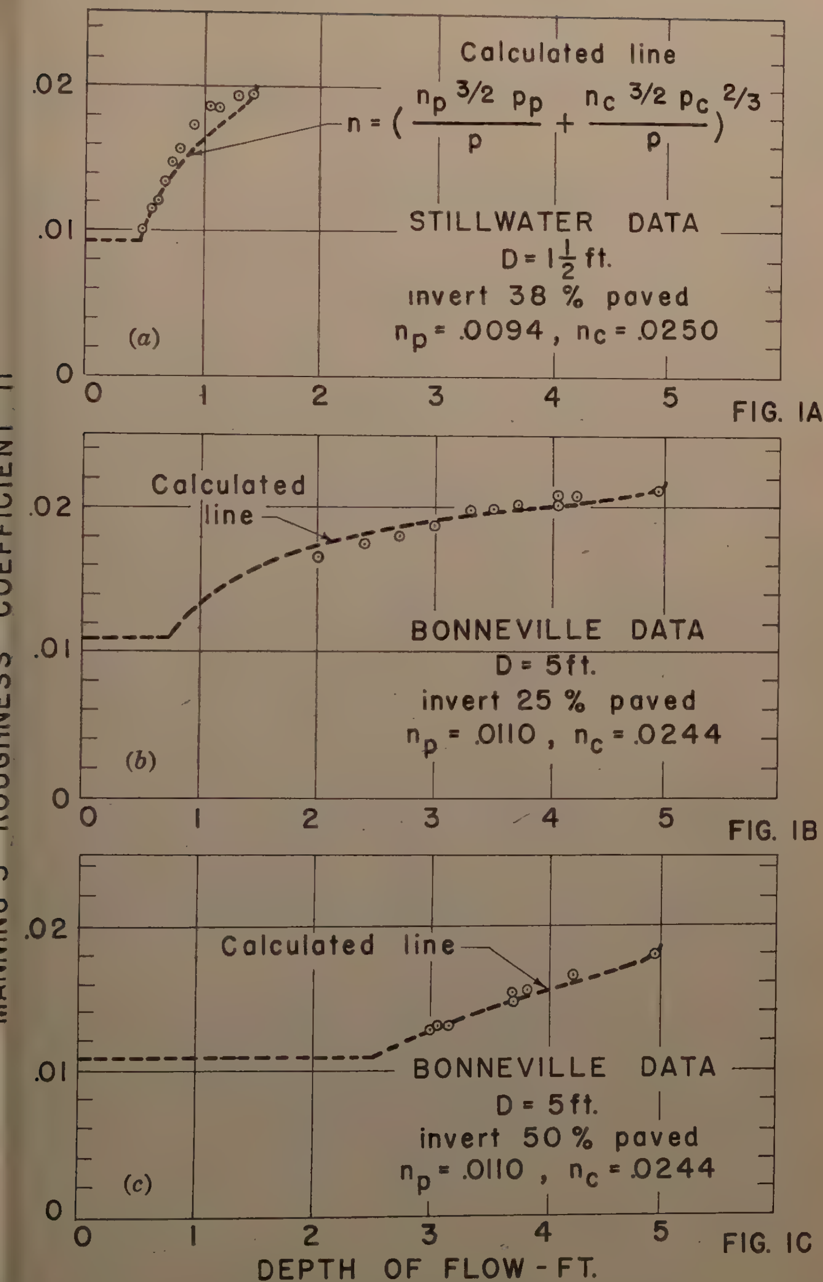
3 "Hydraulic Tests on Corrugated Metal Culvert Pipes," by L. G. Straub and H. Morris, St. Anthony Falls Hydr. Lab., Technical Paper No. 5, Series B, 1950.



G. R1.—VARIATION OF MANNING'S "n" WITH DEPTH FOR CHANNEL FLOW IN PAVED INVERT, CORRUGATED PIPE.

authors' assumed value of 0.011 for the paving. This value was used in the su sequent calculations.

The value of n_c used in the calculations for the 5-ft diameter pipe was t average of the values obtained in the channel flow tests on the corrugated pi without paving. This value was 0.0244. For the 7-ft diameter pipe no channe flow data were reported so a mid-range value of n_c = 0.0237 was used in th case.

The results of the comparison between the predicted values of n and the a tual values of n for the 5-ft diameter pipes are shown on Fig. R1(b) and (The agreement is good.

For the 7-ft diameter pipe with the 25% paved invert the only comparison th can be made is for the full pipe condition. A calculation of Manning's n yield

TABLE R1.—SUMMARY OF TEST RESULTS OPEN CHANNEL FLOW

Q (CFS)	D (FT)[a]	R (FT)	A (SQ FT)	V (FPS)	S (%)	Water Temp (Deg. Fah)	Re X10^{-3}	n
5.52	0.48	0.266	0.482	11.45	3.57	61[b]	461	0.01
6.43	0.56	0.300	0.595	10.80	3.57	61[b]	507	0.01
7.32	0.61	0.319	0.666	10.85	3.57	59[b]	539	0.01
7.66	0.67	0.342	0.752	10.19	3.57	59[b]	556	0.01
8.25	0.74	0.366	0.852	9.68	3.57	45	467	0.01
9.02	0.81	0.387	0.958	9.42	3.57	59[b]	622	0.01
10.0	0.92	0.414	1.122	8.91	3.57	61[b]	687	0.0
11.2	1.05	0.435	1.300	8.61	3.57	59[b]	736	0.01
12.2	1.13	0.442	1.399	8.72	3.57	46	654	0.01
13.2	1.30	0.436	1.590	8.32	3.57	59[b]	881	0.0

[a] D is depth of flow.
[b] Temperature estimated from reading taken on nearest day.

an estimated value of 0.0209. This agrees well with the actual values o 0.0206 to 0.0211) obtained in the range of Reynold's number compatible with selection of 0.0237 for n_c.

The good agreement between the estimated and actual values for Mannin n indicates that the prediction equation has some merit. It should be useful making n-value estimates for other composite pipes within the range of co ditions reported herein.

The writer wishes to express his appreciation to the authors for mak these pipe-flow data available to the profession and for giving him an oppo tunity to test an idea.

EDDY DIFFUSION IN HOMOGENEOUS TURBULENCE[a]

Discussion by Mikio Hino, Takashi Ichiye and Charles G. Gunnerson

MIKIO HINO.[1]—The author's method of determining the turbulent charac-
istics in channel flows is very simple and skillful, and leads to the success-
verification of the Kolmogoroff similarity principle.
However, this relationship (proved by experiments) can also be derived by
theoretical consideration utilizing the well-known statistical characteris-
of the two-dimensional channel flows. Moreover, the constant in Eq. 21
be shown to be dependent on the roughness of the channel bed.
First of all, the mean sizes of eddies, L, for the fully developed pipe or
nnel flows, are determined only by the ratio y/h, where y and h represent
pectively the distance from the wall and the radious of pipe or the depth of
channel. The reasons are as follows: As the experiment by J. Laufer[2]
proved, the production rate of turbulence and the dissipation of energy are
rly balanced in the range of approximately $y/h < 1/2$, and the turbulent in-
sities referred to the friction velocity $\overline{U}_*$, $\sqrt{\overline{u}^2}/\overline{U}_*$ etc, are functions of y/h.
The production rate is given by

$$\text{Production} = \tau \frac{dU}{dy} = \frac{\rho}{k\,y}\left(1 - \frac{y}{h}\right) U_*^3 \qquad (24)$$

sidering the fact that $\sqrt{\overline{v}^2}$ and $\sqrt{\overline{w}^2}$ are almost proportional to $\sqrt{\overline{u}^2}$, and as-
ing the relationship, $\dfrac{\lambda^2\sqrt{\overline{u}^2}}{\nu L}$ = constant, well established for isotropic tur-
nt flows is also applicable approximately for the flows considered,

$$\text{Dissipation} = A'\nu\,\frac{\overline{u^2}}{\lambda^2} = A\left(\frac{\sqrt{\overline{u^2}}}{U_*}\right)^3 \frac{U_*^3}{L} \qquad (25)$$

refore, by equating Eqs. 24 and 25, we have

$$\frac{L}{h} = B\,\frac{\frac{y}{h}}{1 - \frac{y}{h}}\left(\frac{\sqrt{\overline{u^2}}}{U_*}\right)^3 = f_1\left(\frac{y}{h}\right) \qquad (26)$$

September, 1959, by G. T. Orlob.

Post Graduate Student, Dept. of Civ. Engrg., Faculty of Engrg., Univ. of Tokyo,
n.

"The Structure of Turbulence in Fully Developed Pipe Flow," by J. Laufer, NACA,
174, 1954.

The experiment by J. Laufer[3] shows that Eq. 26 holds for the whole fl range. The relationship given by Eq. 26 is also assumable from the expe mental fact that the Prandtl mixing length divided by h, l/h, is expressed as function of y/h only and independent of the wall roughness and the Reynol number.

Therefore, the rate of energy dissipation, Eq. 22, can be modified as

$$E = U_0 \; g \; S_e = f_2 \left(\frac{y}{h}\right) U_*^2 U_0 \, L^{-1} \; \ldots\ldots\ldots\ldots (2$$

where $\overline{U}_0$ expresses the mean velocity of the flow.

Secondly, since the decaying time, $\int_0^{\infty} R \, \xi \, d\xi$, is proportional to $L/\sqrt{\overline{u}^2}$ $L/\sqrt{\overline{w}^2}$, the diffusion rate defined by Taylor, D_z, is represented by

$$D_z = \overline{w^2} \int_0^{\infty} R\xi \, d\xi = C \, \overline{w^2} \frac{L}{\sqrt{\overline{u^2}}}$$

$$= f_3 \left(\frac{y}{h}\right) U_* L \; \ldots\ldots\ldots\ldots (\;$$

Thirdly, the Lagrangian eddy size L_a, is related to L as follows,

$$L_a = \frac{L}{\sqrt{\overline{u^2}}} \overline{U} = f_4 \left(\frac{y}{h}\right) L \frac{\overline{U}}{U_*} \; \ldots\ldots\ldots\ldots\ldots (\;$$

where $\overline{U}$ represents the mean flow velocity at the point.

Lastly, combining these equations and putting y=h, we have

$$D_z = \text{constant} \left(\frac{U_*}{\overline{U}_1}\right)^{\frac{5}{3}} \left(\frac{\overline{U}_1}{\overline{U}_0}\right)^{\frac{1}{3}} L_a^{\frac{4}{3}} \; E^{\frac{1}{3}}$$

$$= \text{constant} \; C_f^{\frac{5}{6}} \left(1 - \frac{1}{K}\sqrt{\frac{C_f}{2}}\right)^{-\frac{1}{3}} L_a^{\frac{4}{3}} \; E^{\frac{1}{3}}, \; \ldots\ldots (\;$$

where

$$C_f = \frac{2U_*^2}{\overline{U}_1^{\,2}}, \quad k = \text{Kármán constant}$$

$$\overline{U}_1 = \overline{U} \text{ at } y = h.$$

The relationship now obtained shows the effect of the bottom roughn which is not great. However, as the author anticipated, the effect will bec apparent when the wide ranges of the bottom roughness are tested.

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[3] "Investigation of Turbulent Flow in a Two Dimensional Channel," by J. La NACA, TN 2123, 1950.

To check the validity of Eqs. 26 and 29, one needs the experimental results owing the relationships between $\overline{U}$, S_e, h, and L_a.

TAKASHI ICHIYE.[4,b]—In the introduction the author described various kinds definitions of eddy diffusivity from an historical point of view, giving the im-ession that they are based on different concepts on diffusion. Instead, these finitions can be derived in a unified manner from a statistical theory of dis-rsion of a particle or particles which is not necessarily pertinent to the eory of turbulence but is essentially based on the theory of Brownian motion random walk. Such an approach was initiated by Batchelor,[5] who derived the finition represented by the Eqs. 1, 2, and 4 from the theory of "one particle alysis" and the neighbor diffusivity in Eq. 6 from "two-particle analysis" of tchelor and Townsend.[6] As for the latter the validity of Eq. 7 over a wide nge of scales of turbulence and the theoretical explanation have been already en elsewhere.[7,8]

The diffusion of one particle in the turbulent field has been discussed by hors of some standard text books differently from the present paper.[6,9,10] e details can be obtained from these books, but it is worthwhile to repro-ce the discussion for comparison with the present paper.

The standard deviation of the z-coordinate (the direction of which is arbi-ry) σ_z or Z^2 of the particle which is at the origin at the initial time is given

$$\sigma_z = \overline{\left[\int_0^t w\,(S)\,ds\right]^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

which w(t) is the turbulent velocity. (The notations of the paper are pre-ved as far as possible, though they are sometimes different from the con-tional ones.)

By introducing the correlation function $R(\xi)$ (or $R\xi$) this equation is written

$$\sigma_z = \overline{w^2} \int_0^t \int_0^t R(\xi_2 - \xi_1)\, d\xi_1 d\xi_2 \quad \ldots\ldots\ldots\ldots (b)$$

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[a] Florida State University, Oceanographic Institute Contribution No. XXX.

[b] Asst. Prof., The Oceanographic Inst., Florida State Univ., Tallahassee, Fla.

[5] "The Application of the Similarity Theory of Turbulence to Atmospheric Diffusion," G. K. Batchelor, Quarterly Journal, Royal Meteorological Soc., Vol. 76, 1959, pp. -146.

[6] "Turbulent Diffusion," by G. K. Batchelor and A. A. Townsend, Surveys in Me-nics, Cambridge University Press, Cambridge, England, 1956, pp. 352 399.

[7] "Horizontal Diffusion," by F. C. W. Olson and T. Ichiye, Science, Vol. 130, No. 4, 1959, p. 1255.

[8] "On Neighbor Diffusivity in the Ocean," by T. Ichiye and F. C. W. Olson, Deutsche rographische Zeitschrift, (in press).

[9] "Turbulent Diffusion: Mean Concentration Distribution in a Flow of Homogeneous bulence," by F. N. Frenkiel, Advances in Applied Mechanics, Academic Press, N.Y., 3, 1953.

[10] "Viscous Flow Theory: II. Turbulent Flow," by S-I. Pai and D. Van Nostrand, nceton, N.J., 1957.

In this derivation we have assumed the condition of homogeneity that the co relation $\overline{w(t+\xi)\;w(t)}$ is a function of ξ only. After some manipulation of tl double integral this is reduced to

$$\sigma_z = 2\,\overline{w^2} \int_0^t (t - \xi)\, R_\xi \, d\xi \, \ldots\ldots\ldots\ldots\ldots ($$

in which we have assumed no more properties of $R\xi$ than being an even fun tion about .

From general properties of the correlation function $R\xi$ two special cas of (c) can be derived immediately. First, since $R\xi$ should vanish for a lar value of ξ, we can define Lagrangian time scale of turbulence L_t as

$$L_t = \int_0^\infty R_\xi \, d\xi \, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots ($$

when $t >> L_t$, Eq. c is written as

$$\sigma_z = 2\,\overline{w^2}\, L_t\, t - 2\,\overline{w^2} \int_0^\infty \xi\, R_\xi \, d\xi \, \ldots\ldots\ldots\ldots ($$

The last term is constant and thus only the first term in the right hand si becomes important for a large value of ξ. The comparison of Eq. e with t probability density of the particle computed from the solution of the Ficki equation gives the eddy diffusivity equal to $\overline{w^2}\, L_t$. Therefore the definition the eddy diffusivity compatible with the Fickian diffusion or Brownian moti is possible only for the case that $t >> L_t$. This definition leads to the ed diffusivity which is constant in time and space. In some problems of meteo ology and oceanography eddy diffusivity which is variable in time and/or spa has been introduced in order to explain distributions of some properties. Ho ever, such an approach will not give any information about the relationshi between the mode of diffusion and physical nature of the turbulence. Accor ingly, the discussion of the Eq. 10 through 13 seems to be meaningless, b cause they just express the Eq. c in different ways by introducing a fictitio definition of eddy diffusivity.

When t is small compared with L_t, the correlation Function $R\xi$ can be wr ten as

$$R_\xi = 1 - \frac{1}{2}\,\xi^2 \left(\frac{dw}{dt}\right)^2 \Big/ \,\overline{w^2} \, \ldots\ldots\ldots\ldots\ldots$$

Then Eq. c becomes

$$\sigma_z = \left(1 - \frac{1}{12}\, t^2 \lambda^{-2}\right) \overline{w^2}\, t^2 \, \ldots\ldots\ldots\ldots\ldots$$

in which a characteristic time λ is defined by

$$\lambda^{-2} = \overline{\left(\frac{dw}{dt}\right)^2} \Big/ \overline{w^2} = -\left(d^2R_\xi / d\xi^2\right)_{\xi=0} \quad \text{(h)}$$

hen $t \ll \lambda$, Eq. g may be written as

$$\sigma_z = \overline{w^2}\, t^2 \quad \text{(i)}$$

The curves of σ_z-t (or σ_z - x as in the present experiment) can be obtained latively easily from the diffusion pattern of dye or discrete particles. Eq. e dicates that the eddy diffusivity $D = D_z(\infty) = \overline{w^2}\, L_t$ can be determined from the ope of σ_z - t curve for $t >> L_t$. Eq. i may be used for estimating $\overline{w^2}$ from e same curve in the neighborhood of t = 0. Then the value of $L_\infty = U\, L_t$ can obtained from D and $\overline{w^2}$ thus determined.

This process of determining the eddy diffusivity does not need any informa- on or assumption about a functional form of the correlation function $R(\xi)$ as the case in the paper. Instead, some authors derived the functional form of ξ) from the curves of σ_z - t obtained by experiments,[9] though direct mea- rements with hot wire technique give more accurate results. Actually the nctional form assumed in Eq. 15 is not valid near $\xi = 0$, because the gradient $(\xi)/d$ becomes discontinuous at $\xi = 0$. The casual inspection of the picture fluorescein dye pattern in the paper reveals that the spread of the dye is not actly expressed by the Eq. 16 near the origin.

The isotropy of the turbulence is not confirmed yet in a shear flow which is basic flow in the present experiment. Even when the turbulence is isotropic, e apparent coefficient of eddy diffusion determined by the procedure similar the one described above or used in the paper is not so. The longitudinal efficient was found to be much larger than the lateral one in the flow of a pe[11] and in an open channel.[12] It seems to be worthwhile to compare the re- lt of the present experiment with those of Elder,[12] since both were obtained an open channel.

Representation of Kolmogoroff's similarity principle may be done in several ys. In a two-particle analysis the relation similar to Eq. 21 has a well- nded background for a representation of this principle, when neighbor dif- sivity F and initial separation l_o are taken instead of $D_z(\infty)$ and L_a. (See tchelor[5] and Ichiye and Olson).[8] On the other hand, in a one-particle prob- n as in the present paper, Eq. 21 expresses only a dimensional relation and physical meaning is obscure. This kind of relation is valid only in a range large wave numbers where the dissipation of turbulence is caused by vis- us forces (Batchelor's equilibrium range). In this range the similarity princi- is expressed by

$$\nu = \epsilon^{1/3}\, \eta^{4/3} \quad \text{(j)}$$

which ν is the molecular viscosity and ϵ is dissipation rate of energy. The racteristic length η is called Kolmogoroff's microscale which corresponds proximately to the minimum wave number of the equilibrium range.

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11 "The Dispersion of Matter in Turbulent Flow through a Pipe," by G. I. Taylor, ceedings, the Royal Society, Vol. A223, 1954, pp. 446 468.

12 "The Dispersion of Marked Fluid in Turbulent Shear Flow," by J. W. Elder, rnal of Fluid Mechanics, Vol. 5, No. 4, 1959, pp. 544-560.

The other difficulty in the application of Eq. 21 is the arbitrary definition of E. In a shear flow this quantity should include frictional velocity or shearing stress at the wall of a channel which in turn includes the molecular viscosity. These factors may be implicitely included in S_e of Eq. 22. But then the constant of Eq. 21 loses its nature as a universal constant. Aside from these theoretical difficulties, the scatter of experimental points of D_z against $E^{\frac{1}{3}} L_z$ in Fig. 11 seems to be too large to ensure the relation of Eq. 21, though Fig. may represent the so called 4/3-power law well.

From the energy spectrum theory we can construct another relation between the eddy diffusivity and the scale of turbulence, which cannot necessarily be claimed to be unique. In an inertial subrange[13] where the dissipation occurs by inertia terms, the energy spectrum E(n) (in which n is a wave number) is given by

$$E(n) = \alpha\, \epsilon^{2/3}\, n^{-5/3} \ldots\ldots\ldots\ldots\ldots\ldots (\text{k})$$

(Batchelor's[13] equation [6.5.1], hereafter indicated by [B6.5.1]).

Put n_0 the wave number of the eddies which contain most parts of energy. Then at n_0, E(n) has a maximal value

$$E_0 = \alpha\, \epsilon^{2/3}\, n_0^{-5/3} \ldots\ldots\ldots\ldots\ldots\ldots ($$

The energy spectrum for a range of small wave numbers may be different different flow system but since we are concerned only with the integrated form of this spectrum the contribution from this range is small. Thus we may put

$$E = E_0\, (n/n_0)^m \quad \text{for} \quad 0 < n < n_0 \ldots\ldots\ldots\ldots (\text{m})$$

in which m is a positive constant. Under the isotropic condition

$$\overline{w^2} = \frac{1}{3} \int_0^\infty E(n)\, dn \ldots\ldots\ldots\ldots\ldots\ldots ($$

Substituting the value of E(n) of Eqs. k and m into Eq. n we have

$$\overline{w^2} = \beta\, \epsilon^{2/3}\, n_0^{-2/3}, \quad \beta = \frac{1}{3}\alpha \quad \frac{3}{2} + \frac{1}{m+1} \ldots\ldots\ldots ($$

The correlation function $\overline{w(t+\xi)\; w(t)}$ is the same as the second equation (B3.4.4). Then the combination of equations (B.3.4.9) and (B.3.4.21) gives

$$\overline{w^2} \int_0^\infty R_\xi\, d\xi = \pi\, (4U)^{-1} \int_0^\infty n^{-1}\, E(n)\, dn \ldots\ldots\ldots ($$

13 "The Theory of Homogeneous Turbulence," by G. K. Batchelor, Cambridge University Press, Cambridge, England, 1953.

which U is the mean current introduced in order to change the original dis-
nce integral into time integral of the present definition of eddy diffusivity.
ain substituting Eq. k and m into Eq. p we have

$$D_z(\infty) = \gamma\, U^{-1}\, \epsilon^{2/3}\, n_0^{-5/3}, \quad \gamma = \frac{1}{4}\pi\,\alpha\left(m^{-1} + \frac{3}{5}\right) \ldots\ldots (q)$$

e scale of turbulence L_∞ can be obtained by

$$L_\infty = U D_z(\infty)\,(\overline{w^2})^{-1} = \gamma\,\beta^{-1}\, n_0^{-1} \ldots\ldots (r)$$

om Eqs. q and 3 the similarity principle is expressed by

$$D_z(\infty) = \beta^{-1} U^{-1}\, \epsilon^{2/3}\, L_\infty^{5/3} \ldots\ldots (s)$$

In a shear flow the mean velocity U may be related with ϵ, but it is impor-
nt that the eddy diffusivity defined from Eq. e is dependent on 5/3 power of
size of the energy-containing eddies in a flow with a constant mean velocity.
The results of the experiments are quite important and interesting, though
erpretation and arrangement of the data may be revised.

Acknowledgment: The author is indebted to Mr. C. G. Gunnerson for his
vice in preparation of the manuscripts.

CHARLES G. GUNNERSON,[14] F. ASCE.—The author's extension of the
our-Thirds Law" shown on his Fig. 8 and 12 is an important contribution.
s statement concerning the significance of eddy diffusion in the problem of
ste assimilation by the receiving water of a large coastal bay immediately
ggests the discharge of Los Angeles' Hyperion Treatment Plant effluent into
nta Monica Bay.[15]

There are several difficulties in applying the eddy diffusion equation:

$$k = e\, L^{4/3} \ldots\ldots (1)$$

ich is a restatement of the author's Eq. 7 and of the trend equations shown
Fig. 12. As the author indicates, there are a variety of definitions of the
le of the phenomena, L. The trend lines on Fig. 12 indicate a range of about
orders of magnitude for the correlation coefficient, e. Finally, it is ap-
rent from the literature cited by the author and others[16,17,18] that there is
universally accepted characterization of horizontal diffusion phenomena and,

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14 Civ. Engr., Bur. of Sanitation, City of Los Angeles, Calif.
15 "Sewage Disposal in Santa Monica Bay," by C. G. Gunnerson, Transactions, ASCE,
. 124, 1959, pp. 823-851.
16 "Diffusion in Bikini Lagoon," by W. H. Munk, G. C. Ewing, and R. R. Revelle,
nsactions, Amer. Geophysical Union, Vol. 30, No. 1, 1949, pp. 59-66.
17 "Horizontal Diffusion in the Sea," by J. Joseph and H. Sender, Deutsche Hydro-
phische Zeitschrift, Vol. 2, No. 2, 1958, pp. 49-77, (translation by G. I. Roden).
18 "Diffusion of Sewage Effluent in an Ocean Current," by N. H. Brooks, Proceed-
s, First Internl. Conf. on Waste Disposal in the Marine Environment, Univ. of Cali-
nia, Berkeley, Calif. (in press).

therefore, the definition of the coefficient of eddy diffusivity, k. Addition difficulties lie in making adequate observations.

A logical method for predicting the behavior of a waste discharge involve the introduction of a suitable tracer into the receiving water. The choice of th tracer depends on local conditions.[19] Fluorescein dye has been used in San Monica Bay and has been released either at a constant rate or intermittentl The latter provides a convenient time scale. Additional information has bee

FIG. G1.—DYE DIFFUSION STUDY IN SANTA MONICA BAY. NOTE APPARENT LA OF RELATION BETWEEN DIFFUSION, CURRENTS AND WAVES.

obtained from the trajectories and dispersion patterns of the more than 50 drift cards which were released from various stations in 1955-56. The gene al agreement of the results of these diffusion studies with other work has be previously reported.[20]

[19] "Tracer Methodology and Pollutional Analyses of Estuaries," by E. A. Pears Proceedings, First Internl. Conf. on Waste Disposal in the Marine Environment, Un of California, Berkeley, Calif. (in press).

[20] "Studies on Eddy Diffusion in Santa Monica Bay, California," by R. B. Tibby a C. G. Gunnerson, Transactions, Amer. Geophysical Union, Vol. 39, No. 3, 1958, p. 5

A comparison of the dye stream in the author's Fig. 5 with photographs of a ich larger dye stream in the ocean reveals the range of variation which ap-
es to diffusion phenomena.

FIG. G2.—DYE DIFFUSION STUDY. EACH SLUG IS ABOUT 350 FT LONG.

The photographs were taken from an airplane of a diffusion experiment in a Monica Bay.[21] A 10% solution of fluorescein was discharged from a skiff nored about 5 miles off shore. The dye was fed at a 12 gal per hr rate and

1 "Summary Report, Oceanographic Investigation of Santa Monica Bay," Bur. of tation, Los Angeles, Calif., 1956.

released intermittently at a depth of about 4 ft on a schedule of 10 min-ON an
5 min-OFF.

The experiment was carried on for about 7 hr. During this time, the curren
underwent one rapid right-angle change in direction. However, the velocity re

FIG. G3.—DYE DIFFUSION STUDY. THE DYE PERSISTED FOR OVER 3 HR.

mained essentially constant at about 1/3 knot. Accordingly, the dye "slug
were about 350 ft in length. The widths of the slugs were obtained from ve
tical photographs which included 2 boats, the positions of which were know
No measurements were taken within 500 feet of the source.

Fig. G1 shows the dye pattern about 1 hr after the abrupt 90° change in cu
rent direction had taken place. Neither the current shift nor the lateral diff

on of the dye appeared to be related to the wind waves or swell.

Fig. G2 shows the variety of patterns which were found. Some of the varia-ons may be due to the boat's swinging on its anchor.

Fig. G3 is an oblique taken during the time when the dye stream was rela-ely stable and when the measurements of lateral diffusion were made. The ht-colored area just beyond the zone where the dye disappeared is the sea-

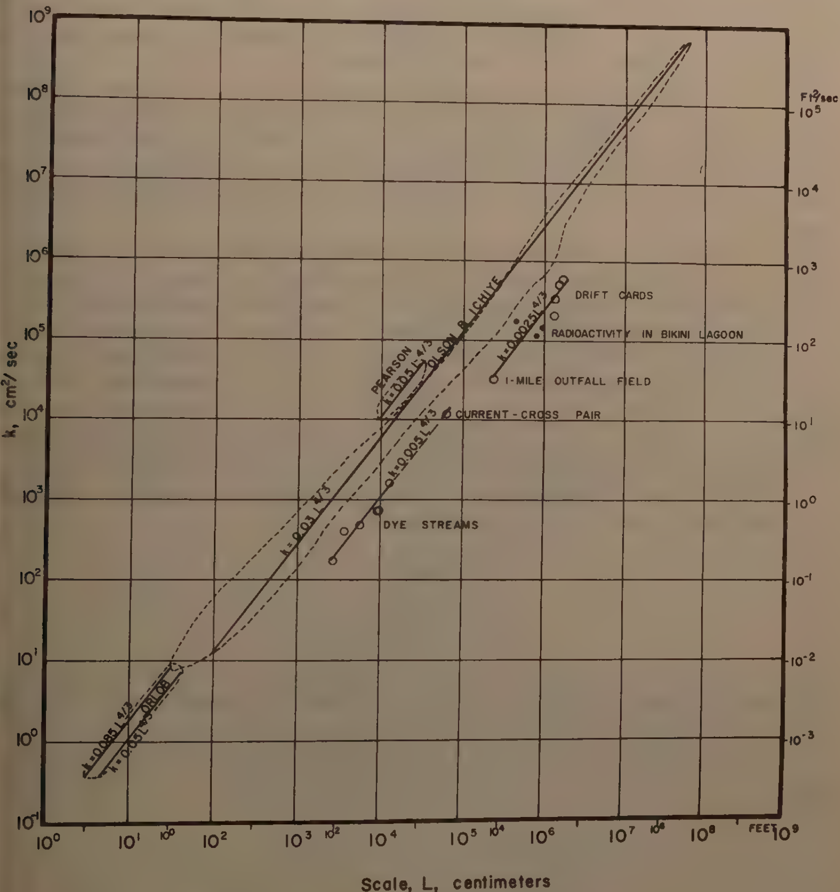

FIG. G4.—RESULTS OF VARIOUS INVESTIGATORS' DETERMINATION OF CORRELATION IN EDDY DIFFUSION EQUATION, $k = eL^{4/3}$.

rd edge of the inshore water mass associated with the discharge of Hyperion uent and of cooling water from nearby steam plants.[15]

The widths of the stream determined from vertical photographs were oothed and used for the diffusion calculations and were assumed to include ut 95% of the stream.

The detailed results of these diffusion studies have been previously report-5 and are included in the author's Fig. 12. Taken by themselves, those re-s are reasonably consistent.

It should be noted that Fig. 12 is typical of a number that have appeare since Inoue[22] in 1950 first assembled an array of published coefficients of edd diffusivity. He found the correlation coefficient in c.g.s. units for the writer' Eq. 1, to be, e = 0.01, although the data showed a scatter of as much as thre orders of magnitude.

Several subsequent investigators have fitted their results to Inoue's curv and found reasonable agreement.[23,24] A few, including the author and Olso and Ichiye,[25] have determined those curves (more precisely, those values c e) which best characterize selected data. This selection is important, for i determines the validity of the final formulation.

A partial summary of the published curves showing the relation of the co efficient of eddy diffusivity to the scale of the phenomena is shown on Fig. G4 The light dashed lines surrounding the curves of the various investigators in dicate the extent and scatter of their data. The coefficients (e) are in metri units, although conversion to English units may be made from the secondar scales.

Some new data are included. The current cross pair value is derived fro observations made during bacteriological testing of Hyperion effluent. Th field was tagged by both dye and current crosses whose center of gravity wa at about the 4-ft depth.[21] The 1-mile outfall field value is based on the sprea of the field from the 1-mile outfall at Hyperios.[15] The drift card values ar based upon analyzing drift card returns reported by Stevenson, Tibby, an Gorsline,[26] using data from groups of stations which showed shore recoverie essentially normal to assumed streamlines. The data on radioactivity in Biki Lagoon[16] are included to show their similarity to drift card observations.

All of these coefficients of eddy diffusivity were determined by means of th formulation of Munk, Ewing, and Revelle[16]

$$k = \frac{\sigma_2^{\,2} - \sigma_1^{\,2}}{2\left(t_2 - t_1\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2$$

where k is the horizontal coefficient of eddy diffusivity in cm^2/sec, and σ_1 an σ_2 are the values of the standard deviation of the concentration of a tracer alor a line perpendicular to the flow at t_1 and t_2, respectively.

The measured widths of the various streams (dye, current crosses, Hyperic effluent) were assumed to contain about 95% of the tracer. Thus, the measure width, u equals 4 σ.

Substituting, Eq. 2 becomes

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22 "The Application of Turbulence Theory to Oceanography," by E. Inoue, (in Japa nese), Journal, Meteorological Soc. of Japan, Vol. 28, No. 11, 1950.

23 "On the Eddy Diffusion of Punices Ejected from Myojin Reef in the Southern S of Japan," by H. Hanzawa, Records of Oceanographic Works in Japan, Vol. 1, No. 1953, pp. 18-22.

24 "Discussion, The Measurement and Calculation of Stream Reaeration Ratio," E. A. Pearson, Proceedings, Seminar on Oxygen Relationships in Streams, Taft Sa Engrg. Center, USPHS, Cincinnati, Ohio, 1958, pp. 43-45.

25 "Horizontal Diffusion," by F. C. W. Olson and T. Ichiye, Science, Vol. 130, 195 p. 1255.

26 "The Oceanography of Santa Monica Bay, California," by R. E. Stevenson, R. Tibby, and D. S. Gorsline, Allan Hancock Foundation, Univ. of Southern California, L Angeles, Calif., 1956; Fig. 82, Stas. 3430, 3436, 3437; Fig. 89, Stas. 3512, 3513, 151 3519; Fig. 95, Stas. 3940, 3945, 3946, 3951; Fig. 96, Stas. 4025-4032, incl.

$$k = \frac{w_2^{\,2} - w_1^{\,2}}{32\,(t_2 - t_1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The scale of the phenomena is taken as:

$$L = \frac{w_1 + w_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

It is significant that for lateral diffusion where no boundary restraints ex-
ed, the curve $k = 0.005\ L^{4/3}$ gives a reasonable fit. Where the flow was

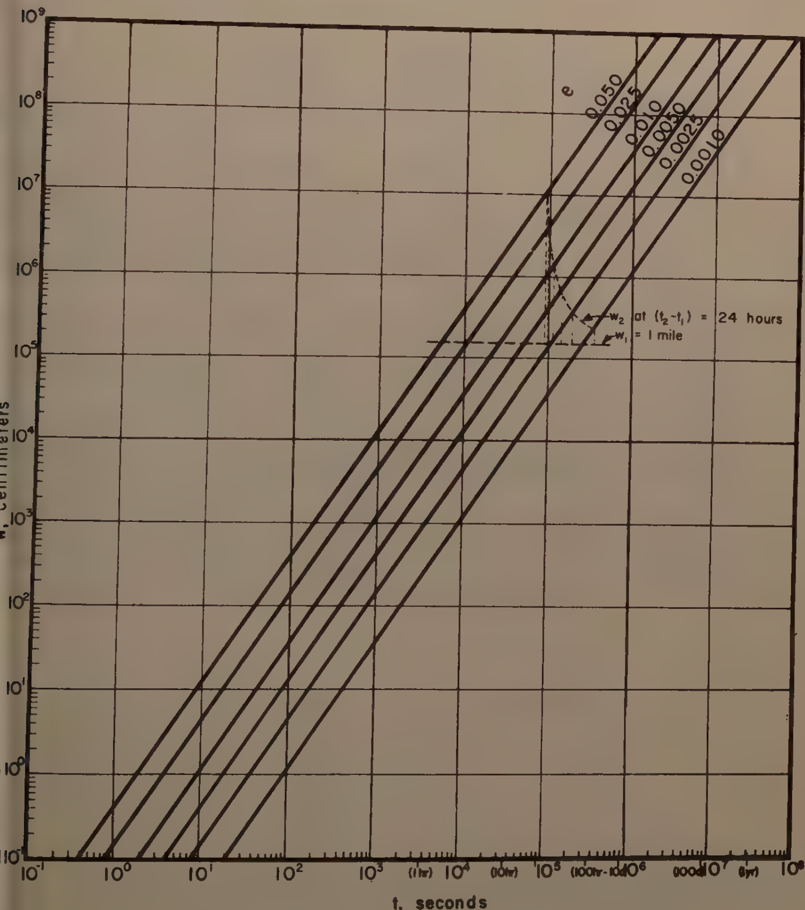

FIG. G5.—SOLUTION OF EDDY DIFFUSION EQUATIONS,

$$k = eL^{4/3} = \frac{\sigma_2^{\,2} - \sigma_1^{\,2}}{2\,(t_2 - t_1)}. \quad \text{SEE TEXT.}$$

bounded on one side by the shoreline, the curve $k = 0.0026\ L^{4/3}$ more accurately describes the data.

Reference to Fig. 4 shows the wide variation in the reported values of th coefficient, e, of Eq. 1. As noted above, the lower values may reflect boundar conditions. The explanation of the higher values determined by the author ma lie in the difference in nature of the turbulence in his laboratory channel an that turbulence in the stratified ocean. Undoubtedly, some of the difference are due to the variety of approaches which have been employed.

In any event, the importance of eddy diffusion in waste disposal problem lies in the feasibility of predicting the spread or dilution of a pollutant fro this phenomenon. It is appropriate to examine the effects of assuming variou values of e in such a prediction.

From the writer's Eq. 1, 2, and 4

$$e\left(\frac{w_1 + w_2}{2}\right)^{4/3} = \frac{w_2^2 - w_1^2}{32\left(t_2 - t_1\right)} \qquad (5)$$

Fig. G5 shows a family of curves for Eq. 5 with values of e of from 0.001 t 0.050, somewhat less than the ranges indicated on Fig. G4. The effect of ap plying various ranges of times to the lateral dispersion ($w_2 - w_1$) was examine throughout a large range of values by means of an IBM 650 computer.[27] Fro this it was determined that the functions are adequately represented by th curves shown.

Fig. G5 may be used to indicate the width w_2 when w_1 and the elapsed tim (t_2=t_1) are known. In the example shown, if $w_1 = 1.6 \times 10^5$ cm = 1 statute mil and a value for e of 0.005 is assumed, the width w_2 after 24 hr will be 7.0 10^5 cm or 4.3 miles.

The 1-mile width was chosen since it is a reasonable first approximation the initial width of the field from the new 5-mile effluent outfall of the Hyperi Treatment Plant. It is expected that initial dilution in the rising column wi be about 100:1, and subsequent dilution will be by horizontal eddy diffusion. Similarly, the elapsed-time figure of 24 hr is based upon current observatio made in Santa Monica Bay[21,25] and represents a typical travel time to shor It is of interest to estimate the dispersion of the field as it travels to shor thus:

w_1	$(t_2 - t_1)$	e	w_2
1 mile	24 hours	0.0010	1.5 miles
		0.0025	2.5 miles
		0.0050	4.3 miles
		0.010	8.5 miles
		0.025	28 miles
		0.050	72 miles

Weighing the indicated values of w_2 against experience, it is possible to assi probable limits to e.

27 Personal communications from D. E. Madden, IBM Corp., Los Angeles, Cali 1949.

The field from the existing 1-mile outfall widens from 1 mile in width ached after about 4 hr from the outfall to 2 miles in width after 24 hr (t_2 - = 20 hr). Diffusion is to the seaward side only. Thus the value of 0.0025 for s probably too low.

The value of 0.010 for e gives a value for w_2 of 8.5 miles. Currents in nta Monica Bay average about 0.3 knot[15] indicating a movement of about 6.3 tute miles in 24 hr. A field of this sort would occupy a sector with an angle about 60°. Observations of water mass movements in the Bay do not support sumption of a lateral dispersion this large. Thus e must be less than 0.010.

Obviously, more observations are required to characterize the dispersion a sewage field. The 5-mile outfall will be in operation in the early part of 60, and some operating data will then become available. However, it is prob-e that the entire field will be submerged most of the time due to mixing with d bottom water and thus acquiring the same density as intermediate depth ters. There are difficulties in sampling such a field, and a number of ex-sive surveys at all seasons are required for definitive work.

Meanwhile, a value for e of 0.005 in Eq. 5 can be expected to permit rea-nable approximations of the lateral diffusion of the effluent field. Prelimi-ry computations indicate that it is satisfactory for predicting the areal ex-t of bacterial pollution.[28]

An evaluation of boundary effects is always required in computations of persion of wastes. The existence and nature of clearly distinct water masses he oceans[29,30] suggests that hydrographic boundaries may be essentially effective as land masses in providing lateral restraint to an ocean current.

On a scale which is important in waste disposal, say 0.1 to 10 miles, it is dent that observations of diffusion processes may profitably be extended. propriate studies might include analyses of the effects of currents, density erences between water masses, vertical stability, and other physical fac-s upon eddy diffusion. The author's laboratory channel may give some in-mation. What happens, for example, when particles of a slightly different e or specific gravity are used? How would two distinctly different streams ract in the channel?

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8 "Predicting Bacterial Pollution in Sea Water," by C. G. Gunnerson, unpublished y, 1958.

9 "The Oceans," by H. U. Sverdrup, M. W. Johnson, and R. H. Fleming, Prentice-, Englewood Cliffs, N.J., 1946.

0 "Allgemeine Meereskunde," by G. Dietrich and K. Kalle, Borntraeger, Berlin, .

THE HYDRAULIC DESIGN OF SPILLWAY BUCKETS[a]

Discussion by M. B. McPherson and Ernest E. Brodbeck

M. B. McPHERSON,[1] M. ASCE.—The principal purpose of this discussion present a comparison of performance between the slotted-type and solid-e roller bucket. A comprehensive solid bucket study covering a wide range parameter values has been reported for a radial exit at 45°, approach slopes 1 on 1 and 1 on 2, with flow uniformly distributed over the channel width.[2]

The "sweepout" condition is identical with a "free-jet," the term used in the id-bucket study. The authors report that bed arrangement was not critical the incidence of the "sweepout" condition. Limited tests with a solid bucket, ng both a movable bed at different elevations and a fixed floor, indicated no asureable affect on surface profiles or performance due to bed arrangement.

With a solid bucket a "pulsating surge" was obtained for large values of 'R (in which h_1 is the total available head relative to the bucket invert and R he bucket radius). A "pulsating surge" is a pronounced vertical, unsteady tion of the standing wave downstream from the bucket, and appears to have ne regime features in common with the "diving flow" condition for slotted kets.

Nearly all of the original solid bucket tests were performed with a falling water. With a rising tailwater, cessation of a "free-jet" requires a higher water level than for the commencement of a "free-jet" on a falling tailwater. ppears that the ratio of the rising to falling tailwater limits might be less 1 1.1 for h_1/R greater than about[3] 3. A similar hysteresis effect is prob- associated with both the "pulsating surge" and the "diving flow" tailwater its. For the "sweepout" tests the tailwater was "lowered slowly" and for "diving flow" tests the tailwater was "raised slowly." One would expect sistence of the "sweepout" condition at higher tailwater levels on a rising water and persistence of the "diving flow" condition at lower tailwater levels falling tailwater. Whether or not the 0.2 ft. added to the "sweepout" depths the 0.5 ft. subtracted from the "diving flow" depths of the slotted bucket lel data might include compensation for tailwater reversal effects is diffi- to say. However, if performance between the two limits can be assured, issue of rising or falling tailwater is not pertinent.

The use of a variable datum, the tailwater elevation, in nearly all graphical sentations by the authors, obviates the need for any trial and error steps

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[a] October, 1959, by G. L. Beichley and A. J. Peterka.

[1] Prof. of Hydr. Engrg., Civ. Engrg. Dept., Univ. of Illinois, Urbana, Ill.

[2] "A Study of Bucket-Type Energy Dissipator Characteristics," by M. B. McPherson M. H. Karr, Proceedings, ASCE, HY 3, June, 1957. (Corrections: HY 4, August, , p. 57. Discussion by E. A. Elevatorski, HY 5, October, 1957, p. 33. Discussion by . Dougherty, HY 1, February 1958, p. 43.

[3] Paper 1832, October, 1958, p. 41.

in the use of their data for design. For the purpose of comparison it is neces sary to use a fixed datum. Fig. M1 has been prepared using the same datu as for the solid bucket study, the bucket invert. The flow parameter include the flow per unit width, q, in cubic feet per second per foot. The term h_b i identical with B and h_2 with T. The slotted bucket model data were taken fro a USBR publication.[4] The term T_{min} equals "sweepout" depth plus 0.2 ft. (es timated minimum tailwater level for roller action on a falling tailwater). Fo a solid bucket and a falling tailwater the approximate limit before a "free-jet is reached is at about h_b = 0.2 h_2. The average-fit T_{min} curve arbitraril drawn for the slotted bucket points indicates a generally higher tailwater re quirement than for the solid bucket, as noted by the authors. The term T_{ma} equals "diving flow" depth minus 0.5 ft. All values of T_{max}, up to and incluc ing those for "maximum capacity" flows, have been plotted in Fig. M-1; curv (c)-(c) is an approximate envelope of the largest "maximum capacity" (i. "design capacity") flow points. The range between the T_{min} curve and curv (c)-(c) is limited, but this fact does not reflect adversely on the recommenda tion that slotted buckets may be particularly suited for lower ranges of tailwate depths. In the range between these curves the depth in the bucket, h_b = B, obviously much greater for the slotted bucket. It could be demonstrated th the surge height A is less for the slotted bucket. In using the model data[4] was taken as the difference between the headwater pool water level and th bucket invert. (A plot similar to Fig. M-1, but for "sweepout" and "divi flow," is available.)[3]

Fig. M2 is analogous to Fig. 15. The maximum h_1/R for a solid bucket, avoid a "pulsating surge" with a falling tailwater, is represented by line S It appears that a smaller radius solid bucket may be used for q-paramet values exceeding about 0.045 (F_1 at bucket invert as high as about 8, from tab in Fig. 5, reference 2). Smaller radii are indicated for slotted buckets with parameter values less than about 0.045 (F_1 at bucket invert greater than abo 8), representative of comparatively higher h_1 values. The solid bucket tes were directed towards applications with low medium head spillways; the slott bucket appears to be more suited to high head installations.

The flatter and more quiescent water surface immediately above and dow stream from a slotted bucket would constitute a decided advantage in certa applications. Also, satisfactory performance has been demonstrated for t slotted bucket with a movable bed only 0.05 R below the apron lip, which co responds to an elevation about 0.1 R below a 45° solid radial bucket lip. It questionable whether a solid bucket bed at 0.1 R below the lip would be sati factory under all conditions, particularly at low tailwater levels; a bed elev tion at the same level as the invert (0.29 R below the lip) has been recommen ed for general usage.[2]

While it is acknowledged that "unsymmetrical spillway operation... (mak the solid) bucket undesirable in some installations," is it to be inferred th this same limitation is not shared by the slotted bucket? Indeed, can eith type of bucket be used with other than uniformly opened crest gates? Fr the information given in the paper the reader has little basis for agreeing th

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[4] a. "Slotted and Solid Buckets for High, Medium and Low Dam Spillways," Progre Report III, Section 7, Bur. of Reclamation, Hydr. Lab. Report No. Hyd.-415, Denv July 1, 1956. b. "Hydraulic Design of Stilling Basins and Bucket Energy Dissipator U.S.B.R. Engrg. Monographs, Denver, No. 25, September, 1958. (Both 2-a and 2 b co tain tabulations of slotted bucket test data.)

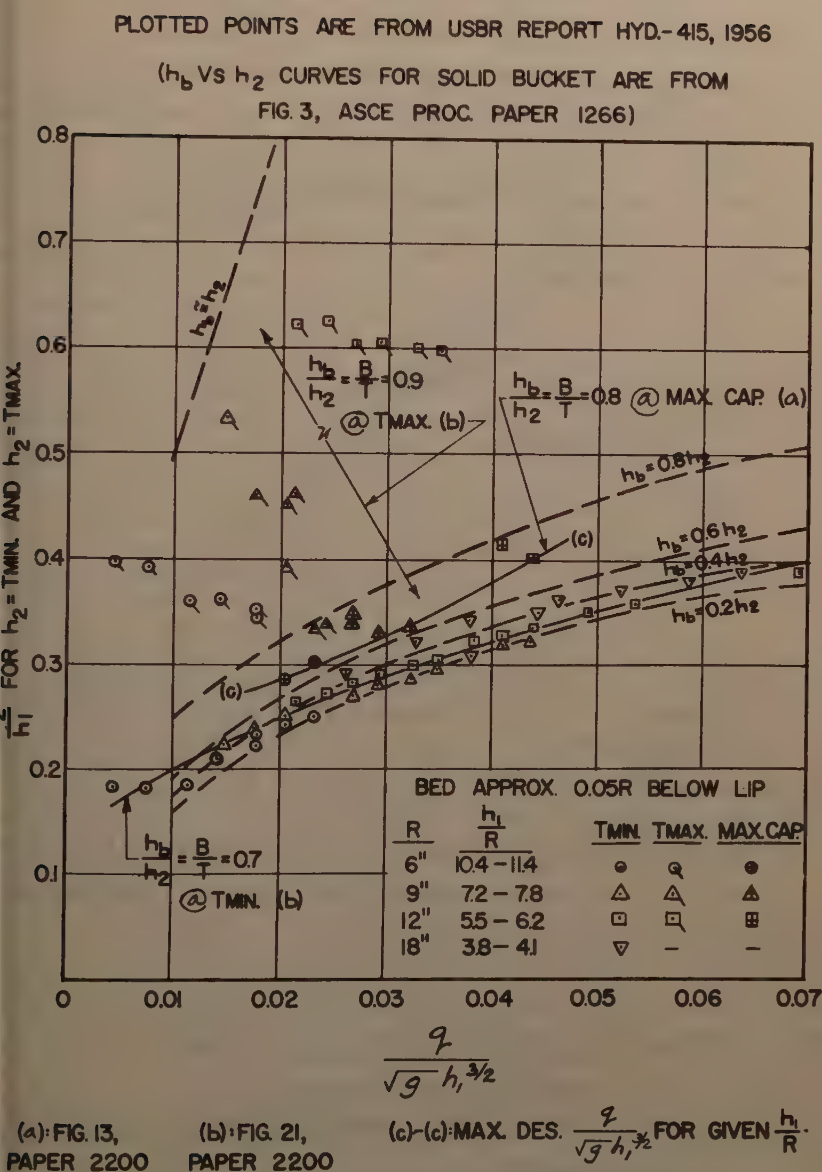

FIG. M1.—SLOTTED BUCKET VS. SOLID BUCKET CHARACTERISTICS.

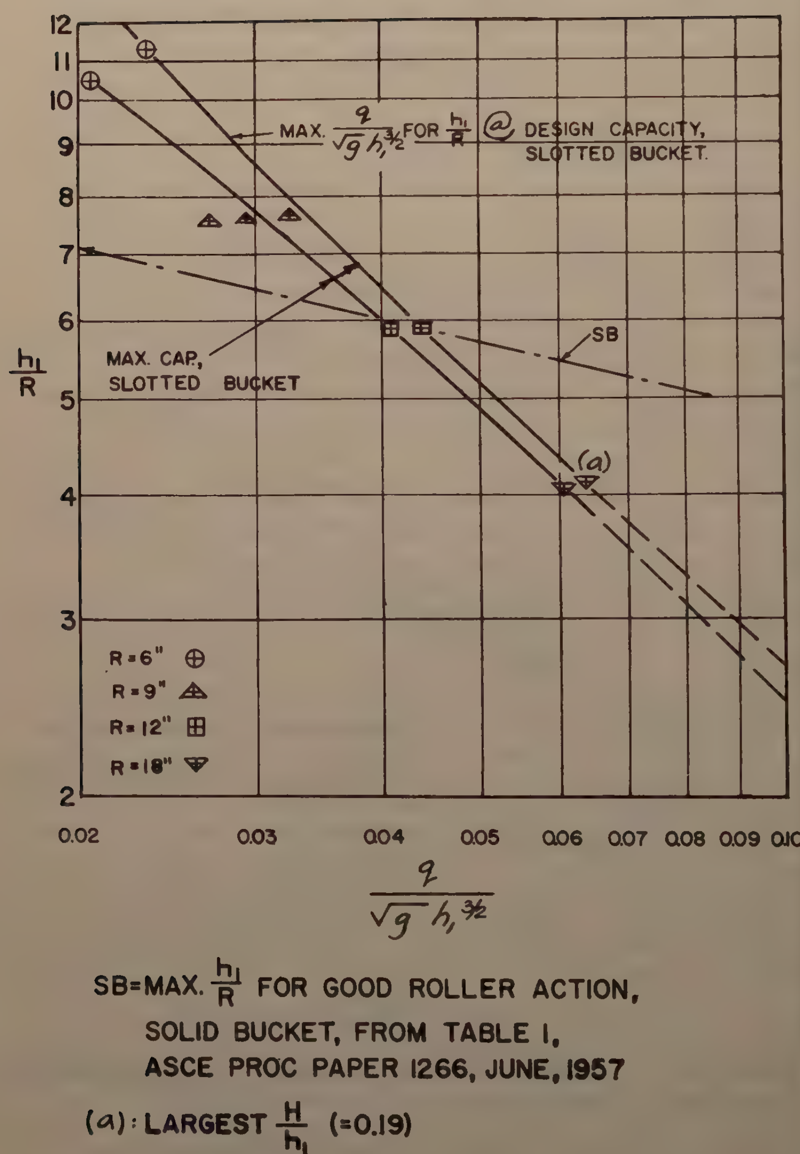

FIG. M2.—MINIMUM ALLOWABLE BUCKET RADIUS.

sts showed the slotted bucket to be superior to the solid bucket in all re- ects" since the promise set forth in the Synopsis, that the slotted bucket "is npared with a solid-type bucket," was not fulfilled. Specifically, what solid ket tests were performed, with what bed conditions, and where can the data acquired? Data from the Grand Coulee Dam model should not be used for nparison since the exit was not radial. The writer is impressed with the lity of the model tests reported, but cannot agree that the tests covered "a nplete range of bucket sizes and tailwater elevations." Comprehensive data flow-tailwater-roller relationships between the limits T_{min} and T_{max} are available. Large values of the q-parameter have been tested for only small 'R and small values of the q-parameter for large h_1/R (or corresponding ameters). Fig. 21 purports to be rather universally applicable, but it ap- rs that only X = 5 ft was used in all the tests in which the surge height, A, s measured. Since all the tests were performed with an approach slope of on 7, what is the permissable extrapolation of the results to other slopes?

The curves in Fig. 22 appear to be identical with those in Fig. 15, pages 3-3, reference [5]. However, in the latter a few points for Shasta Dam and and Coulee Dam (prototype) have been included, and the ordinate is Z (fall m reservoir headwater level to stilling basin floor), with the effective head the theoretical floor velocity equal to (Z - H/2) rather than the (H + h) in . 22. There appears to be a possible inconsistency between these figures. vever, there is good agreement between Fig. 1 of reference [6] and Fig. 22 estimated depth and velocity on a spillway face. The curves of Fig. 15, erence 5, have been modified by Ven Te Chow, M. ASCE,[7] so that the esti- ed floor velocity may be read directly.

n Table 4 the value of F applies to conditions at the shifting tailwater ele- on. Unfortunately, the conjugate depth in Table 4 has been computed using F. This is misleading. The values of D_1 and V_1 used to solve F_1 should aken at the bucket invert for an equitable comparison of tailwater require- ts. The equation at the bottom of page 34 cannot be applied otherwise. For le M-1 the average of T_{min} = 67 ft and T_{max} = 71 ft, or 69 ft, at the design (901 cfs/ft in Table 3) was used to set the bucket invert at El- 3045. The e other values of T are the difference between the tailwater elevations n in Table 3 and El. 3045. The conjugate depths shown in Table M-1 should vide a better basis for comparison.

eferring to Table M-1 and Fig. M-2 or Table 3 and Fig. 15, it should be d that the calculations for design flow conditions given for Angostura Dam stitute an extrapolation beyond the test data. In Table 3 a minimum R of t is estimated; a minimum R of about 50 ft is indicated in the extrapolated e of Fig. M-2 (h_1/R = 2.7 maximum). How can a 40-ft radius for Angos- Dam be the actual size of the existing structure, the "recommended" ra- in Table 3 and then be changed arbitrarily to 47 ft in Table 4, for the de- discharge? The actual invert of Angostura Dam is at El. 3040 for which design discharge q-parameter would be 0.097 and h_2/h_1 would be 0.53, with 74 ft, a conjugate depth of 67 ft and h_1/R of 3.5 using the actual 40-ft

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"Hydraulic Design of Stilling Basins: High Dams, Earth Dams, and Large Canal ctures (Basin II)," by J. N. Bradley and A. J. Peterka, Proceedings, ASCE, HY 5, ber, 1957, pp. 1402-7 and 1403-3.

"Air Entrained in Fast Water Affects Design of Training Walls and Stilling Ba- " by D. B. Gumensky, Civil Engineering, Vol. 19, No. 12, December, 1949, p. 35.

"Open Channel Hydraulics," by Ven Te Chow, McGraw-Hill, N.Y., 1959, pp. 382-

bucket radius. As may be seen in Fig. M-1, there is no realistic basis fo comparing the invert 3040 and 3045 from the standpoint of maximum allowab tailwater, without gross extrapolation of curve (c)-(c) well beyond the graph.

TABLE M1 (Disc. 220

Revision of Tables 3 and 4, Paper 2200, for Angostura Dam with Crest at El. 3157.2 and Bucket Invert at El. 3045, Bucket Radius 40-feet.**

q,cfs/ft.		901***	657	365	146
h_2,ft. (=T)		69*	61	50	39
Z,ft. (a)		153.1	146.0	136.5	125
V_T,ft./sec.		92.5	91.2	89.5	87.
V_1/V_T (a)		0.97	0.96	0.95	0.9
V_1,ft./sec. (c)		89.7	87.5	85.0	78.
D_1,ft. (c)		10.05	7.51	4.29	1.8
h_1,ft. (b)		134.6	126.5	116.3	97.
h_2/h_1		0.51	0.48	0.43	0.4
h_1/R		3.4	3.2	2.9	2.4
$\frac{q}{\sqrt{g}\, h_1^{3/2}}$		0.10	0.081	0.051	0.0
F_1 (c)		4.98	5.62	7.23	10.
D_2,ft. (c) (T_{conj})	q	66	56	42	26

Notes: (a) See Fig. 15, Paper 1402, October, 1957.
(b) Net, or effective, total head relative to bucket invert.
(c) At bucket invert.
*Ave. of T_{min} and T_{max}, max. Q, Table 3, Paper 2200
**Actual Bucket Radius, 40-ft.; Invert at 3040
***Design Flow

It is hoped that the authors will present pertinent details and a summary results obtained with the slotted bucket models for Angostura Dam and t Superior-Courtland and Cambridge diversion dams,[8] as well as similar info mation for whatever solid bucket tests have been performed.

PROBABLE CORRECTIONS

The word "not" should be stricken from the first line of page 22.

In Figure 15 for R = 12-in. the flows should be 3.5 and 3.25 rather than t 3.00 and 2.0 given. The units of flow should be cfs/ft. rather than cfs. (P reference 2).

8 "Hydraulic Energy Dissipators," by E. A. Elevatorski, McGraw-Hill, N.Y., 19 pp. 175-178.

The terms to the right of the 1/2 in the equation at the bottom of page 34
ould be in brackets or parentheses.

On the last line of page 25 the relation beneath the square-root should in-
de (H + h) rather than (H + H).

ERNEST E. BRODBECK,[9] F. ASCE.—The authors merit the appreciation of
se concerned with the hydraulic design features of energy dissipators. The
iter has always felt that a paper presenting hydraulic design criteria based
theoretical analysis or model test data should be augmented by an example
wing its practical application. This has been accomplished, in part, by the
hors.

In the design of hydraulic structures, particularly large dams, it is essen-
l that the designing engineer envision operating conditions and evaluate their
ential effect on the performance of the completed structure. In this regard,
authors have given cognizance to such conditions by stating, "Before adopt-
a design, all factors which might affect the tailwater range should be in-
tigated; that is, large or sudden increases in spillway discharge and effects
discharges from outlet works or power plants." However, a more detailed
cussion of "all factors" should have been included to assure proper recog-
on of operational features and conditions as they affect bucket performance.

With the above in mind, the writer offers the following comments with re-
d to bucket performance vs. operating conditions. For example, consider
ungated overfall spillway with a flood flow discharge. Under such condi-
is, the tailwater rise or fall is generally compatible with the spillway over-
v. Therefore, the design data for minimum and maximum tailwater as pre-
ted in the paper would be directly applicable. However, with a gated spill-
structure, unusual operating conditions could be created. During a period
reservoir rise it may be incumbent upon the project operator to open the
lway gates for flood flow release. Required sudden gate openings with in-
ased discharges could result in a condition wherein the magnitude of the
lway overflow could be considerably greater than the ability of the existing
water to maintain or confine spillway overflows to roller action in the
ket. As a result of the high discharge and deficient tailwater depth, sweep
would occur as shown in Fig. 7A.

Similarly, a sudden decrease in discharge by closing of spillway gates, when
ailwater is at a high stage, could result in a diving jet similar to that shown
Fig. 3 and 7 C. Such a condition could prevail as a result of a sudden de-
ase in spillway flow being released upon a tailwater depth established by a
eding greater flow release.

s both sweep out and diving flow represent undesirable flow conditions
a potential for channel scour, it is incumbent upon the hydraulic engineer
nalyze carefully the spillway gate operating schedule so as to maintain over-
s and tailwater depths within the operating range required to assure proper
er bucket action. In order to accomplish this, a knowledge of the rate of
and fall of tailwater depth vs. discharge is required. Also, the problem
egradation or downstream channel changes which may affect tailwater
es should not be overlooked.

here remains another design feature which is often not given too much
ideration. This feature concerns itself with the establishment of the height

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Hydr. Engr., North Atlantic Div., Corps of Engrs., New York, N.Y.

radius of slot C apparently does not prove that the corresponding incipie cavitation parameter would be little affected. Since "field experience sho cavitation below a slot with such a 12:1 convergence" and since the lowe boundary pressure was found below the slot it is implied that damage dow stream from the slot is necessarily associated with the minimum bounda pressure. However, it is not clear whether sharp or rounded upstream a downstream corners or some combination are at issue. With the W.E. rounded upstream corner tests, the initiation of cavitation occurred within t void of the slot. If cavitation is also found to commence within the slot, w the sharp upstream corner of the Type C slot, the minimum boundary pressu occurring at the end of the convergence may not be a significant criteria f anticipating incipient cavitation. As to field damage below the slot, perha vapor produced in the slot implodes upon reaching the low pressure zone the P.I. It is not clear whether or not the author has conducted any cavitati tests on the Type C slot. What are the dimensions of the slot with "the ou wardly offset, 1-inch-rounded corner with a 12:1 convergence"? Does t term "cavitation pressures" used in the same sentence mean actual inciden

TABLE M2

Velocity greater than (f.p.s.)	Offset height, y, (in.)	$\frac{H - }{V^2/2}$
45	1/16	0.85
45	1/8	1.4
30	1/4	2.0
20	1/2	2.4

of cavitation in the tests? If so, at the point of minimum observed bounda pressure?

Into-the-Flow Offsets.—The writer is particularly interested in Fig. 2 showing "Head-Velocity Relationship for Incipient Cavitation—Abrupt, Int the-Flow Offsets." The sketch appears to be for an open channel but the te apparatus used was probably a closed system similar to the Fig. 4 facility. is assumed further that H is the piezometric head at an upstream referen point, relative to the elevation of the top of the offset, and that V is the ave age velocity of flow in the lesser cross-section downstream from the offse If the vapor pressure head is designated h_x and points from Fig. 20 are co verted into a pressure-type parameter, it is found that the parameter is fair constant beyond a certain minimum velocity (Table M2).

Using 31 piezometer taps, Williams[5] made extensive pressure distributi measurements for conditions including abrupt into-the-flow offsets in a close conduit and an open channel. The closed conduit tests covered 13 values of ranging between 0.06-in. and 1/4-in. for an approach conduit height of 0.500 0.833 ft, with a maximum ratio of offset to conduit height of 0.04. The approa section, between a transition piece leading from a large entrance tank and th offset, was 12-in. long; the conduit was 6-in. wide. Available flow limited a

[5] "Study of Misalignments in an Open Channel and a Closed Conduit," by J. C. Wi liams, Jr., M.S. Thesis, Lehigh Univ. Library, Bethlehem, Pa., May, 1952.

roach velocities to a maximum of 24 fps. His results can be expressed in rms of the parameter given in the following table, in which H is the approach ezometric head, h_x is the minimum piezometric head (3/64-in. downstream om face of offset) and V_O is the average approach velocity. Values of the rameter approached and reached a constant as the approach velocity was in-reased. Sample asymptotic values from his Fig. 12 are given in Table M3.

TABLE M3

Offset height, y, (in.)	$\frac{H - h_x}{V_o^2/2g}$
1/16	0.66
1/8	1.06
1/4	1.12

V (velocity above offset, as in Table M2 was used stead of the approach velocity V_O, the above pa-meter values would be increased by a maximum 9% (for the 1/4-in. offset and 0.50-ft. approach ight). Referring to Table M2, it is to be noted at the incipient cavitation parameter values are eater than the pressure parameter values by illiams, typical of abrupt flow-disturbances. One ason the pressure parameter values are lower is cause the true minimum pressure cannot be mea-red. The minimum pressure occurs at the offset rner, the initial point of flow separation. The pressure gradient immediate- downstream from the offset is too steep for satisfactory extrapolation of otted boundary pressure distribution data to the offset corner.

Holl[6] has reported on cavitation tests in water tunnels with triangular and rcular-arc offsets. The triangular elements were 90° isosceles with one leg rpendicular to the floor and the other facing downstream. The triangular ele-ents were mounted on a flat plate with an airfoil shaped leading edge. The ate was installed in the central plane of the circular tunnel test section. For hase 1" the test elements were placed 11-in. from the leading edge of the ate ("Forward Station") and at 37-in. ("Center Station"). "One would expect at the pressure field would be dependent upon the location of the element in boundary layer," and "Also, the shape of the boundary-layer velocity pro- e would influence the pressure field." Boundary-layer velocity profile mea-rements were made for all triangular element tests. In Table M4 are ap-oximate incipient cavitation parameter values for one series of tests (Phase with velocities of 30 to 60 fps, where H is the approach piezometric head, is the vapor pressure and V_O is the approach velocity. If one assumes that streamline near the point of separation of the triangular elements is simi- in curvature to that for a rectangular-shaped offset, a direct comparison in order. The general magnitude of parameter values in Tables M2 and M4 in reasonable agreement. As the boundary layer becomes small compared h the offset height, the cavitation parameter in Table M4 increases. It may seen that a given ratio of offset height to boundary layer thickness (1.1) is ociated with a different cavitation parameter value at the two stations. The io of boundary layer displacement thickness to momentum thickness is 1.40 the Forward Station and 1.33 for the Center Station, a boundary layer shape ameter. "For given values of velocity and boundary-layer shape parame-, the incipient-cavitation number increases with relative height of rough-s. Furthermore, the relative height of roughness appears to be the most nificant variable for describing the incipient-cavitation number."

Williams[5] reasoned that the local velocity approaching an offset was a con-lling factor, with the boundary-layer velocity profile thereby governing the

[6] "The Inception of Cavitation on Isolated Surface Irregularities," by J. W. Holl, ME, No. 59-Hyd-12 (to be published in the ASME Journal of Basic Engineering).

of training walls. The model tests presented in the paper were accomplished in a flume which voided the condition of training wall overflow from tailwater back-up. As the training walls at a bucket are seldom constructed to the maximum height of tailwater, some overflow at this location could prevail. If such overflow were of a significant magnitude, a diving jet adjacent to a training wall could occur with consequent damaging scouring action concentrated at the toe of the wall. Such bucket action could be avoided by raising the wall to such a height so that training wall overflows would be insignificant insofar as the stability of bucket roller action is concerned. Proper operation sequence of spillway gates adjacent to the wall could also alleviate such undesirable flow conditions.

The foregoing is offered in discussion only to emphasize that the final bucket design should be thoroughly investigated and anlayzed in the light of conditions that may prevail during normal and emergency spillway operation.

HYDRAULIC CHARACTERISTICS OF GATE SLOTS[a]

Discussion by A. Thiruvengadam, M. B. McPherson and Warren H. Kohler

A. THIRUVENGADAM.[1]—Fig. 20 shows the head-velocity relationship for incipient cavitation for abrupt offsets into the flow. This datum would be much more useful if the offset is considered as a single isolated roughness protruding into the flow. Then the roughness Reynolds Number would be V z/ν where V is the velocity, z is the offset in feet, and ν is the kinematic viscosity. The relative roughness would become Z/H where H is the depth of flow beyond the offset. The critical cavitation parameter

$$K_i = \frac{H - H_v}{\frac{V^2}{2\,g}} \qquad (1)$$

where H_v is the vapor pressure and g is the acceleration due to gravity. Then Fig. 20 would provide data for the plot of the equation

$$K_i = \left(F\,\frac{Z}{H}, \frac{V\,Z}{\nu}\right) \qquad (2)$$

The writer has plotted this in Fig. T1. If such a relationship could be used for checking the design of stilling basins, the designers would be much benefitted.

The writer records his thanks to N. S. Govinda Rao, F. ASCE, for his suggestions in preparing this discussion.

M. B. McPHERSON,[2] M. ASCE.—The major objective of this discussion is to solicit further clarification and amplification from the author. The broad scope of his excellent paper has obviously required considerable condensation of certain selections.

Gate Slots.—Presentations of pressure distribution profiles are given in terms of the "reference pressure immediately downstream from gate leaf," in Fig. 8 through 15 and in Fig. 17. It appears that the cross section "immediately downstream from the gate leaf" must coincide with the "downstream edge of the slot." The figures in question are presumed to be for two-dimensional tests without a gate leaf. Therefore, the reference point used is somewhat vague. Is the pressure at the reference point representative of an average or normal gradient, away from the gate slot and unaffected by both

[a] October, 1959, by J. W. Ball.
[1] Tech. Asst., Civ. & Hydr. Engrg. Sect., Indian Inst. of Science, Bangalore 12, India.
[2] Prof. of Hydr. Engrg., Civ. Engrg. Dept., Univ. of Illinois, Urbana, Ill.

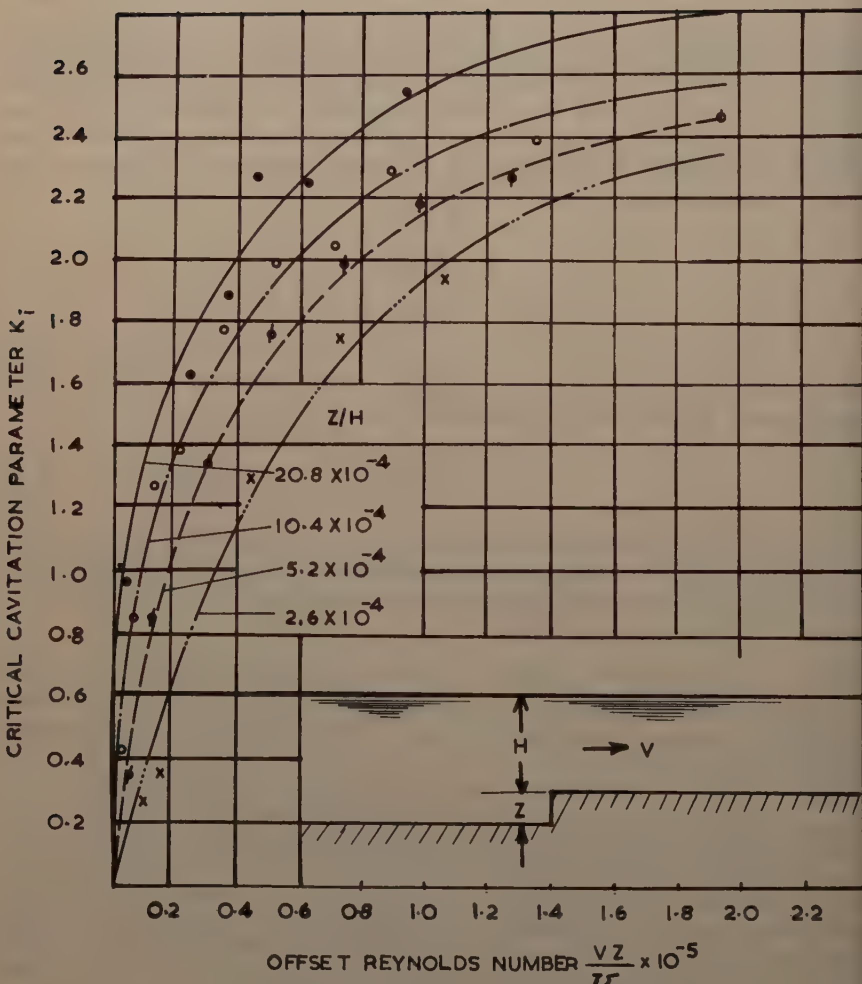

FIG. T1.—RELATION BETWEEN OFFSET REYNOLDS NUMBER AND CRITICAL CAVITATION PARAMETER WITH RELATIVE OFFSET AS THE THIRD PARAMETER.

he leaf wall-aperture and the seat? Was the conduit cross-sectional area aken upstream from the gate slot, at the gate slot, or at some point down-tream from the slot, to compute the "velocity head at reference point"?

In the original data given by Kindsvater[3] for "Slot B" of Fig. 11, the refer-nce point was taken at "an arbitrary upstream control pievometer," and the normal friction grade line" was extended from this upstream control point ast the gate slot position. From Fig. 68 of the reference it appears that the ighest smoothed-curve pressure-parameter value would be about +0.20 and ne lowest about -0.063 (lowest plotted point = -0.070), using the "normal grade ne" at the downstream edge of the gate slot and "the computed average ve-ocity head." What procedure was employed by the author to obtain the +0.19 nd -0.12 extremes given in Fig. 11 for Kindsvater's data? The minimum ressure for Slot B is shown at a distance of about 6.1-in. downstream from ne gate slot in Fig. 11 whereas it is given as about 13.5-in. in Fig. 68 of the eference (that is, at the P.I.).

Incipient cavitation parameter values have been presented by Brown[4] for a 2:1 tapered slot similar to Type B, but for W/D = 1.55 Table M1 is presented.

TABLE M1

Radius at ownstream dge of slot	Taper length, edge of slot to P.1.	Taper offset, total**	$\frac{h_x - h_o}{h_v}$
).0425 W	0.250 W	0.0625 W	-0.36
).0425 W	0.500 W	0.0850 W	-0.29
).0775 W	0.750 W	0.140 W	-0.28
).1050 W	1.000 W	0.187 W	-0.22
).114 W)*	(1.228 W)*	(0.205 W)*	?

*Slot B, Figure 11, W/D = 1.8.
**12:1 Taper plus downstream corner curve.

he last column in Table M1 has been expressed in terms of the author's pa-ameter since the reference head and velocity were "at the slot;" the equivalent h_x was taken as the vapor pressure head by Brown. "The data presented in-cate about the same degree of cavitation for each slot." In the W.E.S. tests, avitation flashes first appeared in the eddies within the slot proper."

In Fig. 11 and 12 it is shown that with W/D = 1.2, a 12;1 convergence and a arp upstream slot corner, the lowest pressure parameter value was decreased ightly from -0.16 to about -0.14 with "no adverse effects" when the down-ream from the P.I. (where the lowest pressure occurs) with a 12:1 conver-ence and a sharp upstream slot corner is about -0.12, as opposed to an in-pient cavitation parameter of about -0.4 indicated by gross extrapolation of e table in the above paragraph. For a rounded upstream slot corner incipient ıvitation reference heads are greater the more the taper offset is reduced. ne fact that the pressure parameter was little changed by omitting the 0.056

[3] Discussion of "Cavitation in Hydraulic Structures—A Symposium," by C. E. Kinds-ter, Transactions, ASCE, Vol. 112, 1947, pp. 101-105.
[4] "Hydraulic Models as an Aid to the Development of Design Criteria," by Frederick Brown, Corps of Engrs., Waterways Experiment Sta., Bulletin No. 37, Vicksburg, ss., 1951, pp. 21-23.

magnitude of the pressure parameter. Unfortunately Williams did not mak velocity traverse measurements and his analysis was purely qualitative. H argued that a given sized offset located near the entrance of a prototype con duit, where the boundary layer is thin, would have a higher pressure parame ter value than if it were located downstream in the zone of established flov The trends in Table M4 appear to lend some credence to this argument.

For truly representative and generalized results, it will probably be neces sary to restrict future offset studies to fully developed boundary layers, make complete velocity profile measurements, and to express the results terms of boundary layer characteristics.

It is hoped that the author will provide the details on the tests represente by Fig. 20, in his closure to the discussions. Also, it is suggested that he de fine the criteria used for delineating incipient cavitation.

An interesting comparison might be made between the circular-arc analys and accompanying incipient cavitation results presented by Holl and tl circular-arc pressure profile data given by Hickox.[7]

TABLE M4

Offset height, y, (in.)	Ratio of y to mean bound. layer thickness	$\frac{H - h_x}{V_o^2/2g}$ (approx.)	Ratio of y to mean bound. layer thickness	$\frac{H - h}{V_o^2/2}$ (appro
1/16	0.425 (1.1)*	0.90 (1.3)*	0.138	0.80
1/4	1.70	1.55	0.552	1.42
1/2	3.40	2.2	1.10	1.8

*By graphical interpolation.

WARREN H. KOHLER.[8]—The information and data given in this paper pro vide a sound and authoritative background as well as a progressive approa and analysis of the gate slot problem. The accuracy and reliability of the tes have been proved by the success of prototype gates which were designed on t basis of data secured by the model tests. This success was not achieved, how ever, on the basis of random application of available data to designs. To a co siderable degree the success of prototype gates is due to the close coordin tion and cooperation which exists between Laboratory and Design Division the Bureau of Reclamation. The designing and building of prototype gate based on small scale models, necessarily involves some changes and compro mises between the ideal conditions in the model and what is practical and o tainable on the prototype from the design, fabrication, and cost standpoint This adaptation requires care, ingenuity, and continued consultation with t

7 "Cavitation in Spillway Tunnel," by George H. Hickox, Fontana Proj. Hydr. Mo Studies, T.V.A. Tech. Monograph No. 68, Knoxville, Tenn., Chapter 4, 1953.

8 Head, Large Gates and Valves Sect., Mech. Branch, Bur. of Reclamation, Denve Colo.

aboratory to avoid deviations from the test models which could seriously af-ect operation of a prototype gate. This projection of the data from a 6-in. nodel to a gate which may be 6, 8, or 10 ft in size does pose a number of de-ign problems. The following comments are primarily intended to point out ome of the design problems involved in translating the model to the prototype.

While not mentioned in the paper, the problem of providing an economical ate design with sufficiently smooth fluidways to avoid local areas of cavita-ion under high-velocity flow posed a design problem. Completely machined luidways would have been hydraulically satisfactory, but the machine work nd extra joints required for machining would have been very expensive. The esign problem was solved by changing gate bodies from the conventional cast esign to welded design. The change avoided the necessity of machining fluid-ays, as the rolled surface of the plates forming the fluidway were smooth nough without requiring additional machining. Some problems in welding gate odies to avoid undue distortion were encountered, but numerous bodies with a luidway plane tolerance of 1/16-in. in 3-ft have been successfully and economi-ally fabricated. The use of welded construction makes curved or other shapes ownstream from the gate slots economically feasible, as bending the plate to uit a template curve is relatively simple and cheap when compared to the cost f machining such a surface.

As a matter of fact, one of the basic reasons for considering a small offset n the downstream side of gate slots was that it would be economically prohibi-ive to try to match the openings of the upstream and downstream bodies of ates exactly. This led to the investigations which showed that a small offset ctually was a hydraulic improvement at the slot. The faring of this offset back the initial fluidway width did, however, pose some problems hydraulically hich the model tests solved.

The elimination of the radius on the downstream edge of the gate slot was articularly helpful from a design as well as a hydraulic standpoint. Offhand, would not seem that reduction in the span width of a gate which is 6 or 7 ft ide by 2 in. would be material, but in the narrow-slot type of gate where only tongue on the side of the gate extends into the slot, the reduction in bending noment amounted to nearly 20% on the critical tongue extensions.

The author speaks of the use of slot fillers, but from the standpoint of pro-otype design, these pose serious problems. If a slot filler does not aline al-nost perfectly with the gate slot so that no edges protrude into the high velocity ow, it can be an even worse problem from a cavitation standpoint than an open lot. Unless the designer is prepared to pay the penalty involved in design and linement costs for providing almost perfect alinement, slot fillers should be voided.

The author describes the improvement which can be achieved by moving the ate slot upstream from the spring point on the bottom of a gate leaf to provide cavitation-free slot for both free discharge and back pressure conditions. he problem in this design is the seal; and while several designs which possi-y might have been successful were investigated, the basic seal problem is ot a simple one. The complication lies chiefly in developing a seal which is ufficiently rugged for high heads and has enough flexibility to seal in three lanes involving right-angle corners. The gates for which this design was rst proposed had to withstand heads in excess of 400 feet. For this reason, ntried and questionable seals were not acceptable, and the conventional ar-angement and seals were used. It may be that some new thinking will result

in a satisfactory seal which can be tried on a lower-head gate. For the tim being, however, the design has fallen victim to the gate designer's axiom, "I you can't seal it, forget it."

CAVITATION DAMAGE OF ROUGHENED CONCRETE SURFACES[a]

Discussion by A. Thiruvengadam

A. THIRUVENGADAM.[1]—This paper deals primarily with the inception of avitation on rough surfaces. The author deserves to be congratulated for his ttempt to correlate the inception pressure with shear velocity which is defiitely a new approach.

The writer wishes to clarify one point. The title of the paper is "Cavitation amage on Roughned Concrete Surfaces." But the paper deals only with the ineption of cavitation. The cavitation inception and cavitation damage are two ifferent aspects of the cavitation phenomenon obeying different physical laws.

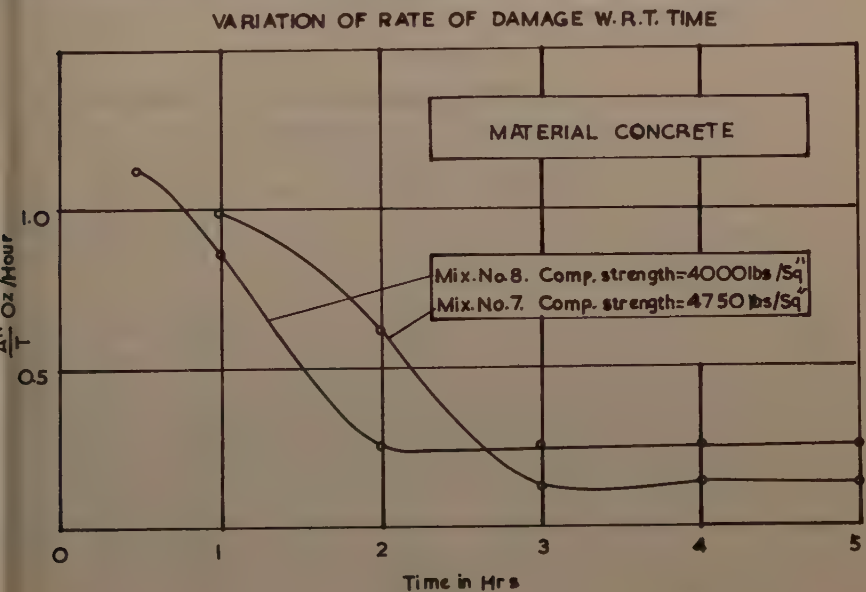

FIG. T1.

The experiments conducted by the writer at the Indian Institute of Science aboratories have clarified certain points on the effect of surface roughness n cavitation damage. The experiments were conducted in a venturi-type cavition tunnel similar to the author's tunnel. Fig. T1 shows that the intensity of amage (rate of weight loss per hour of test) is maximum at the beginning of

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[a] November, 1959, by Donald Colgate.

[1] Tech. Asst., Civ. & Hydr. Engrg. Sect., Indian Inst. of Science, Bangalore 12, India.

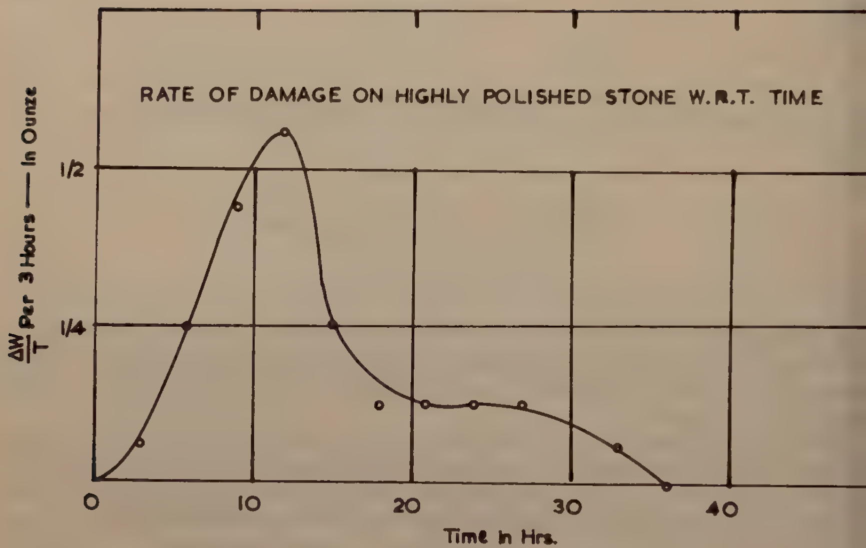
RATE OF DAMAGE ON HIGHLY POLISHED STONE W.R.T. TIME
ΔW/T Per 3 Hours —— In Ounze
1/2
1/4
0
10
20
30
40
Time in Hrs.

FIG. T2.

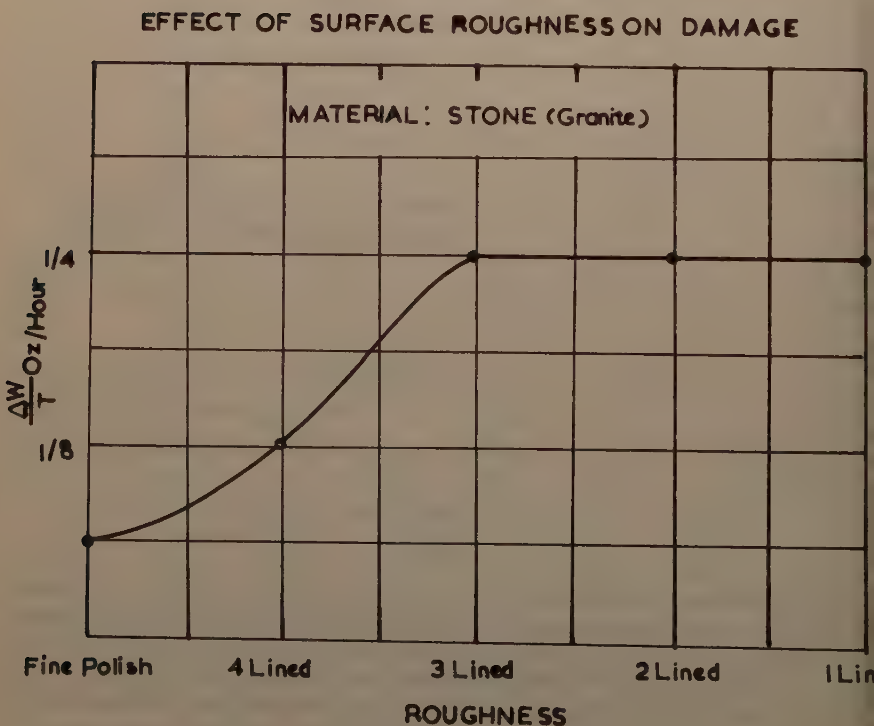
EFFECT OF SURFACE ROUGHNESS ON DAMAGE
MATERIAL: STONE (Granite)
ΔW/T Oz/Hour
1/4
1/8
Fine Polish
4 Lined
3 Lined
2 Lined
1 Lin
ROUGHNESS

FIG. T3.

e test on a natural concrete surface (compressive strength of the concrete is nown in the figure itself). The rate of damage falls down and reaches a con-ant value after a particular time depending on the compressive strength. A ranite block was polished till the surface became glassy in appearance, and sted. The compressive strength of the granite tested was 16400 psi. Fig. T2 nows how the damage develops and falls down after prolonged testing. Then fferent surface roughnesses were produced on the same granite and tested r three hours each. Fig. T3 shows the variation of the rate of damage with e surface roughness (see Table T1 for definition of surface roughness lopted).

TABLE T1

Definition	Description as per public works department code
e line dressed	Chisel dressed so that no portion of the face dressed is more than 1/4 inch from the edge of a straight edge laid along the face of the stone.
wo line dressed	Chisel dressed so that no portion of the face dressed is more than 1/8 inch from the edge of a straight edge laid along the face of the stone.
ree line dressed	Chisel dressed so that no portion of the face dressed is more than 1/16 inch from the edge of a straight edge laid along the face of the stone.
ur line dressed	The surface of the stone is dressed until a straight edge laid along the face is in contact at every point.
nely polished	The surface is polished till the surface becomes glassy in appearance.

From these data, two points are clear. One is that the intensity of cavita-on damage on polished surfaces increases with time and then decreases. The itial increase in damage is due to the progressive cracking on account of the peated collapse pressures of the bubble. The decreasing trend may be due the cushioning out of the shocks by the water filling the pits formed. There-re it is clear that polishing the surface is only a temporary remedy. The cond point is that the roughness as such has no effect on cavitation damage yond a certain limit.

EARLY HISTORY OF HYDROMETRY IN THE UNITED STATES[a]

Discussion by J. W. Johnson

J. W. JOHNSON,[59] M. ASCE.—During the 1860's and 1870's when hydraulic ining activity was at its height in California, several historically important gineering developments occurred. Some of these projects were the first of eir kind in the west, and a few of them were the first of their kind in the rld, all taking place in a relatively small area of the Mother Lode region.[60] name some of the more important, it was here that the first large-scale velopment of reservoirs and canals was made in the United States for min-g, irrigation, and power purposes; the first use in the country of iron pipe der heads of several hundred feet; it was the birthplace of the Pelton wheel, d here occurred the invention of the needle nozzle; in this area was one of e first plants in the west for the generation of electric power under high ads; the first long-distance, high voltage, power-transmission line in the rld; and the first successful long-distance telephone line in the world. any of these historic engineering works are still in operation.

An important contributor to the developments during this era was the well-own hydraulic engineer, Hamilton Smith, Jr., who, in the writer's opinion, ght well have been mentioned in the paper. Smith is best known for his ok[61] "Hydraulics," the background for this treatise being obtained while he s connected with the North Bloomfield Gravel Mining Co. While at North oomfield, Smith became the recognized authority on all matters relating to draulic mining in California. Between August 1871, when he was appointed perintendent of the North Bloomfield company, and 1881, when he left the ited States for mining work in South America,[62] he not only designed and nstructed large dams, pipe lines, and tunnels, but he also conducted impor-it experiments on orifices, weirs, water wheels, and on the flow of water in e lines. Smith's paper, "Water Power with High Pressures and Wrought on Water Pipe," was awarded the American Society of Civil Engineers' omas Fitch Rowland Prize in 1884.[63] The results of his experiments on ifices and weirs still are quoted in present-day hydraulics textbooks.[64] His ta on flow in pipes were obtained by tests conducted on such pipe lines as e Texas Creek and Humbug inverted siphons of the North Bloomfield system[65]

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a January, 1960, by Steponas Kolupaila.

59 Prof. of Hydr. Engrg., Univ. of California, Berkeley, Calif.

60 "Early Engineering Center in California," by J. W. Johnson, California Historical ciety Quarterly, Vol. 29, No. 3, September, 1950, pp. 193-209.

61 "Hydraulics," by Hamilton Smith, Jr., New York, 1886.

62 "Memoir of Hamilton Smith, Jr.," Transactions, ASCE, Vol. XLVI, 1901, pp. 564-

63 "Water Power with High Pressures and Wrought Iron Water Pipe," by Hamilton ith, Jr., Transactions, ASCE, Vol. XIII, 1884, pp. 15-31.

64 "Hydraulics," by G. E. Russell, New York, 1942, p. 112.

65 "Hydraulics," by Hamilton Smith, Jr., New York, 1886, pp. 290-302.

and the Cherokee siphon in Plumas County. To provide more precise data friction losses than was possible with experiments on large pipes, tests we made by Smith on relatively small pipes at the famous New Almaden quicks ver mine in Santa Clara County.

The orifice and weir experiments by Smith were conducted at Columbia H and North Bloomfield and were financed by the three large companies supply water to the mines in that region. The purpose of the experiments was to pr vide precise discharge coefficients for the type of orifice that was used in m tering water to the various mining operations. A complete description of details of the tests is given in Smith's "Hydraulics."

Hamilton Smith also is credited with giving modern tunnel building its sta when he built the mile-and-a-half Malakoff tunnel at North Bloomfield in 18 to explore and work gold-bearing gravel in an ancient stream channel. Pri to construction of the Malakoff tunnel, tunnels generally were excavated fr one end, sometimes from two ends. In the Malakoff tunnel, Smith worked fr both ends but, in addition, sand eight shafts and worked both ways from ea thus giving sixteen faces to work on simultaneously. Examination of Smit personal papers, now in the University of New Hampshire Library, shows t much information on tunnel construction was obtained in 1871 by correspon ence with Edward Frost of Frost Brothers, consulting engineers of Bosto who gave Smith considerable data on costs, rate of progress, and other d obtained in constructing Hoosac tunnel on the Troy and Greenfield Railroad.

With the mounting opposition to hydraulic mining, Smith left California 1881 for a successful mining career in South America, Africa, and Lond Finally he formed a partnership in New York with Henry C. Perkins, who h worked with Smith at North Bloomfield. Smith died on July 4, 1900, at his ho in Durham, New Hampshire. His will provided funds for the library of t University of New Hampshire, which is known as the Hamilton Smith Libra Unfortunately, only a small portion of his personal books and papers is ava able to yield further information on a man who found time to perform class engineering research at a time when the gold output of the mines was the i portant item to most people.

NEW APPROACH TO LOCAL FLOOD PROBLEMS[a]

Discussion by Bernard L. Golding

BERNARD L. GOLDING,[2] M. ASCE.—The author and the Tennessee Valley thority are to be congratulated for the action they have taken in calling the ention of both the public and Congress to this "positive method" of flood conl, that is, the inexpensive method of flood prevention by the establishment of eam encroachment lines or flood plain zoning rather than the expensive thod of flood control dams, levees, etc., after a damaging flood has occurred. ey are also to be congratulated on the action they are taking in the Tennes Valley watershed area in supplying the local communities with the neces-y hydrologic and hydraulic information they need to set up a system of flood in zoning.

The communities of the Tennessee Valley watershed area are fortunate that overnment agency is available in their area to supply them with the data nesary for the establishment of such zoning. As mentioned by the author, in ny states there is no such agency, either Federal, State or local, for which nmunities may go to for help, even though they may recognize the need to e action. However, the lack of such an agency should not deter a community a county from the establishment of such flood plain zoning or regulation. actual hydraulic and hydrologic problems involved in determining the ext of the areas of the flood plain that will be inundated by a particular design d are not as difficult as was indicated by the author, such that a major ncy must be available to solve them. Enough technical information is avail-e from various sources, so that local engineering forces or local engineer forces supplemented by consulting engineers can do the job effectively. st communities already have the necessary legal assistance to draft the essary legislation once the technical problems have been solved. Counties h as Fairfax County in Virginia and Westchester County in New York have d their engineering forces supplemented by consulting engineers to do ef-tive flood plain zoning work in their areas.

Technical information necessary for the solution of hydraulic and hydroc problems is readily available to the engineer from many sources. Disrge information is available from the U. S. Geological Survey who have lished a series of papers on regional flood frequency, in which discharges various frequencies have been correlated with the drainage basin characstics for rivers and streams in a particular area. Discharge information lso available from the Bureau of Public Roads which has made similar renal correlations. Methods for the computation of water surface curves kwater curves) have been presented in many technical publications readily

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a January, 1960, by Herbert D. Vogel.

2 Head, Hydr. Dept., Howard, Needles, Tammen & Bergendoff, New York, N.Y.

available to any engineer. An excellent, simple, easy-to-understand met for computing water surface profiles has been presented by Joe M. Lara, ASCE, and K. B. Schroeder.[3] An entire manual for the computation of wa surface profiles written by the Corps of Engineers is also available.[4] Pap on headwater losses through bridges are also available from both the Geolo cal Survey and the Bureau of Public Roads.

The actual cost of a flood plain zoning program to a community is qu small in comparison to the cost to the community that would result if eve small housing development or several industrial plants constructed on the fl plain were damaged by a flood. Besides the cost of engineering services, only additional expense is the cost of doing the necessary survey work wh generally consists of cross sections necessary for the hydraulic computati and any necessary boundary or real estate surveys necessary to establish actual zones or encroachment lines. Most communities or counties genera have full time survey parties which can do the necessary survey work dur the times when they are not busy on other work. The cost of aerial surv work is not behind the financial capabilities of even small communities an may be used to obtain cross sections and show the extent of zones.

The necessary hydraulic and hydrologic computations done in conjuncti with a flood plain zoning program in small communities will in many instanc provide other indirect benefits. Any existing bridge waterway areas which a too small so that headwater losses through them could cause extensive upstre damage during flood periods will readily become apparent. The results of su computations will also enable local engineers to design adequate bridge wate way areas for new local or county bridges, which in many instances are ba underdesigned in this regard. Often the cost of replacing a single county brid owing to a washout will be only slightly less than the cost of an entire progr of flood plain zoning for a small community.

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3 "Two Methods to Compute Water Surface Profiles," April, 1959.
4 "Manual for Civil Works Construction."

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)c, 2008(EM2)c, 2009(ST4)c, 2010(SM2)c, 2011(SM2)c, 2012(HY4)c, 2013(PO2)c.

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)c, 2041(PL2)c, 2042(PP1)c, 2043(ST5)c, 2044(SA3)c, 2045(HY5)c, 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)c, 2073(IR2)c, 2074(PO3)c, 2075(ST6)c, 2076(HY6)c, 2077(SM3)c, 2078(WW2)c.

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)c, 2098(SA4)c, 2099(EM3)c, 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)c, 2139(PO4)c 2140(SM4)c.

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)c, 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)c, 2179(IR3)c, 2180(HW3)c, 2181(SA5)c, 2182(HY9)c, 2183(SU1)c, 2184(WW3)c, 2185(PP2)c, 2186(ST7)c, 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)c, 2233(PL3)c, 2234(EM4)c, 2235(HY10)c, 2236(SM5)c, 2237(ST8)c, 2238(PO5)c, 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)c, 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)c, 2270(ST9).

DECEMBER: 2271(HY12)c, 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)c, 2324(CP2)c, 2325(SM6)c, 2326(WW4)c, 2327(IR4)c, 2328(PO6)c, 2329(ST10)c, 2330(CP2).

VOLUME 86 (1960)

JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)c, 2349(HY1)c, 2350(ST1), 2351(ST1), 2352(SA1)c, 2353(ST1)c, 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)c, 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)c, 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)c, 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)c, 2391(SM1)c, 2392(PO1)c.

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)c, 2419(WW1)c, 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)c, 2426(HY3)c, 2427(ST3)c.

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)c, 2456(HY4)c, 2457(PO2)c, 2458(ST4)c, 2459(SM2)c.

c Discussion of several papers, grouped by divisions.